Cleveland Innerbelt Bridge

REVISED CALCS FOR NDC 0048

BRIDGE CUY-90-1633 (BRIDGE 6)
SFN: 1809687

E 14TH ST ON-RAMP TO I-90 WB AND FUTURE (CCG3)
MAINLINE I-90 WB OVER E 9TH ST

CUY-90-14.90
PID No. 77332 / 85531

CALCULATIONS HAVE BEEN REVISED TO REFLECT f'c = 11ksi, AND TO COMMUNICATE THAT DESIGN WORKS. NOT ALL PLAN SHEETS WERE UPDATED TO REFLECT THE DESIGNS PROVIDED HERE, i.e. BEARING LOADS, SCREEDS.

HNTB

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Description (Click on Links)

**********NOTE**********
During the course of the design, the bridge numbers was revised. Due to the large volume of output, some of the headers were not updated. For reference purposes, the following is applicable:

CUY-90-1633 = CUY-90-1627

SLAB DESIGN

Calculations For	Innerbelt Bridge No. 6	Job No.	49633	Sheet No.
Made by	AKS	Date	1/11/2011	**1/10/2012**
Checked by	DBT	Date	2/3/2011	**1/16/2012**
Backchecked by	AKS	Date	2/3/2011	**1/16/2012**

LRFD Design - One Course Construction - Summary of Design Criteria and Output — LEFT OVERHANG

Input Parameters Case 1:

Deck & Beam Dimension Input:

Beam Type (Select From Drop Down List)	Concrete Bulb-Tee	Web Thickness, t_w =	8.0000 in	
Beam Spacing, B =	8.833 ft	Top Flange Width, b_f =	48.0 in	
Slab Overhang, O =	3.797 ft	CL Beam to Span (Interior), I =	1.250 ft	
CL Beam to Curb Line, C =	2.297 ft	CL Beam to Span (Exterior), E =	1.250 ft	
Thickness of Concrete Slab Interior Span, T_S =	8.50 in	Slab Thickness @ Critical Sect. in Overhang, T_O =	8.50 in	
Minimum Thickness of Concrete Slab Overhang, $T_{O\text{-}Min}$ =	8.50 in	Thickness of LMC Overlay, T_L =	0.00 in	

Loading Input:

Uniform Weight of Permanent Form, W_F =	0.000 ksf	Noise Wall/Fence Height, N_H =	0.000 ft	
Uniform Weight of Future Wearing Surface, W_{DW} =	0.060 ksf	Noise Wall/Fence CG to Curb, N_{CG} =	0.000 ft	
Increase/Decrease (LL+I) Depending on Loading =	1.00	Noise Wall Weight, N_W =	0.000 klf	
LL Wheel Load, P =	16.0 kips	Wind Pressure, P_W =	0.000 ksf	
Interior Design Positive Moment, M^+_{LL+I} =	6.29 k-ft/ft	Design Span, S = B - 2 x I =	6.33 ft	
Interior Design Negative Moment, M^-_{LL+I} =	3.71 k-ft/ft	Effective Span, S_{eff} = B - t_w =	8.17 ft	Per AASHTO LRFD 9.7.2.3

Concrete & Reinforcement Input:

Assumed Wearing Surface of Slab, T_W =	1.0 in	Concrete Compressive Strength, f'_c =	4.5 ksi	
Positive Moment, Bottom Reinf. Concrete Cover, C_{Vb} =	1.50 in	Yield Stress of Reinforcing, f_y =	60.0 ksi	
Negative Moment, Top Reinf. Concrete Cover, C_{Vt} =	2.50 in	Modular Ratio, n =	7.13	
Unit Weight of Deck Concrete, γ_{conc} =	0.150 kcf	Critical Section for Add'l Reinf., Interior Slab =	10.0 in	Dist. CL Ext. Beam into Interior
Include Sidewalk Dead Load for Interior Design?	No	Length of Sidewalk from FF of Barrier, L_{SW} =	0.00 ft	
Thickness of Sidewalk in Interior Span, $T_{SW\text{-}I}$ =	0.00 in	Thickness of Sidewalk in Overhang, T_{SW} =	0.00 in	Average Thickness

Positive Moment Bottom Reinforcement	#5 bar spa. @	5.50 in	Abar =0.31in^2	dia. = 0.625 in
Negative Moment Top Reinforcement	#5 bar spa. @	5.50 in	Abar =0.31in^2	dia. = 0.625 in
Additional Reinforcement in Overhang	#5 bar spa. @	22.00 in	Abar =0.31in^2	dia. = 0.625 in

Output:

Interior Design:

Moment Capacity — Adjust Input Values to Meet Design Adequacy for these Critical Moments and Stresses! — Over-reinforced ?

Positive Moment :	ϕM_n =	15.97 k-ft/ft	**OK! > Mu**	M_u =	11.90 k-ft/ft	**OK! < 0.42**
Negative Moment :	ϕM_n =	15.97 k-ft/ft	**OK! > Mu**	M_u =	7.39 k-ft/ft	**OK! < 0.42**

Crack Control

Positive Moment :	spa =	5.50 in	**OK! < Max spa**	Max. spa =	11.27 in	
Negative Moment :	spa =	5.50 in	**OK! < Max spa**	Max. spa =	14.57 in	

Overhang Design:

Part 1) Design Case 1, at Base of Barrier : Horizontal Vehicular Collision Load - Extreme Event Limit State

Moment Capacity	ϕM_n =	21.29 k-ft/ft	**OK! > Mu**	M_u =	20.08 k-ft/ft	**OK! < 0.42**

Part 2) Design Case 1, at Critical Section, Overhang Side : Horizontal Vehicular Collision Load - Extreme Event Limit State

Moment Capacity	ϕM_n =	21.43 k-ft/ft	**OK! > Mu**	M_u =	20.22 k-ft/ft	**OK! < 0.42**

Part 3) Design Case 1, at Critical Section, Interior Span Side : Horizontal Vehicular Collision Load - Extreme Event Limit State

Moment Capacity	fM_n =	21.44 k-ft/ft	**OK! > Mu**	M_u =	15.05 k-ft/ft	**OK! < 0.42**

Part 4) Design Case 3, at Critical Section, Overhang Side : The loads that occupy the Overhang - Strength Limit State I

Moment Capacity	ϕM_n =	19.54 k-ft/ft	**OK! > Mu**	M_u =	2.26 k-ft/ft	
Crack Control	spa =	5.50 in	**OK! < Max spa**	Max. spa =	57.32 in	

Part 5) Design Case 3, at Critical Section, Overhang Side : The loads that occupy the Overhang - Strength Limit State V

Moment Capacity	ϕM_n =	19.54 k-ft/ft	**OK! > Mu**	M_u =	2.19 k-ft/ft	
Crack Control	spa =	5.50 in	**OK! < Max spa**	Max. spa =	57.32 in	

Part 6) Design Case 3, at Critical Section, Interior Span Side : The loads that occupy the Overhang - Strength Limit State I

Moment Capacity	fM_n =	19.54 k-ft/ft	**OK! > Mu**	M_u =	12.46 k-ft/ft	
Crack Control	spa =	5.50 in	**OK! < Max spa**	Max. spa =	9.39 in	

Part 7) Design Case 3, at Critical Section, Interior Span Side : The loads that occupy the Overhang - Strength Limit State V

Moment Capacity	fM_n =	19.54 k-ft/ft	**OK! > Mu**	M_u =	10.11 k-ft/ft	
Crack Control	spa =	5.50 in	**OK! < Max spa**	Max. spa =	9.39 in	

Part 8) Compute Additional Reinforcing Overhang Cutoff Length Requirement

Total Length of Add'l Reinforcement past CL Beam =			**20.0 in**			**OK > 25 bar diameters**
Moment Capacity	fM_n =	17.07 k-ft/ft	**OK! > Mu**	M_u =	16.94 k-ft/ft	**OK! < 0.42**
Moment Capacity	fM_n =	15.97 k-ft/ft	**OK! > Mu**	M_u =	11.04 k-ft/ft	**OK! < Max spa**
Moment Capacity	fM_n =	15.97 k-ft/ft	**OK! > Mu**	M_u =	9.14 k-ft/ft	**OK! < Max spa**

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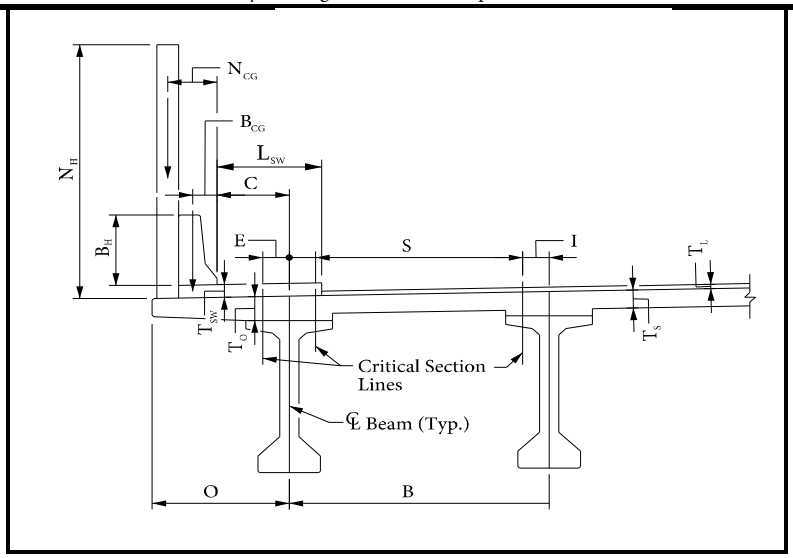
LRFD Design - One Course Construction - Summary of Design Criteria and Output LEFT OVERHANG



INTERIOR DESIGN OF CONCRETE SLAB:

1) Dead Load Moments

W_{DC} =Total Uniform Load of Slab, Perm. Form and Overlay

$W_{DC} = (T_S + T_L) / 12 * \gamma_{conc} + W_F =$ 0.11 ksf

W_{sw} = Uniform Load of Sidewalk

$W_{sw} = (T_{sw}) / 12 * \gamma_{conc} =$ 0.00 ksf

Note: Overlay ignored in W_{DC} if sidewalk within interior bay.

Load Factor, γ_{pDC} =	1.25	For W_{DC}	*AASHTO Table 3.4.1-1 Table 3.4.1-2*
Load Factor, γ_{pDW} =	1.50	For W_{DW}	
Load Factor, γ_{LL} =	1.75	For M_{LL+I}	
Resistance Factor, ϕ =	0.90	For Flexure	*AASHTO 5.5.4.2.1*
Load Modifier, η_i =	1	For All Loads	*AASHTO 1.3.5*

Positive and Negative Moment

$M_D = (W_{DC} + W_{DW} + W_{sw}) S^2 / 10 =$ 0.67 k-ft/ft

Equations are derived from $w l^2 / 8$ x 0.80 continuity factor

$M_{U(D)} = (\eta_i \gamma_{pDC} (W_{DC} + W_{sw}) + \eta_i \gamma_{pDW} W_{DW}) S^2 / 10 =$ 0.89 k-ft/ft

2) Live Load Moments

Positive Moment $M_{U(LL+I)} = \eta_i \gamma_{LL} M^+_{LL+I} =$ 11.01 k-ft/ft

Negative Moment $= \eta_i \gamma_{LL} M^-_{LL+I} =$ 6.49 k-ft/ft

3) Design of Reinforcement

Unit Width of Concrete Slab, b = 12.00 in

Positive Moment

$A_{s\ PRV'D}$ =	0.676 in^2/ft	
Effective Depth, $d = T_S - T_W - C_{Vb} - d_{bar}/2$ =	5.688 in	
$M_{U(D+LL+I)} = M_{U(D)} + M_{U(LL+I)}$ =	11.90 k-ft/ft	
$\rho_{PRV'D}$ =	0.00991	
$a = A_{s(Bott)} f_y / 0.85 f'_c b$ =	0.884 in	
$\phi M_n = \phi A_s f_y (d - a/2)$ =	15.97 k-ft/ft	**OK! > Mu**
$R_N = M_U / (\phi b d^2)$ =	0.41 ksi	
$\rho_{REQ'D} = .85 (f'_c / f_y) \{1 - [1 - 2 R_N / (.85 f'_c)]^{.5}\}$ =	0.00722	
$A_{s\ REQ'D} = \rho_{REQ'D} b d$ =	0.493 in^2/ft	
Req'd Bar Spacing $= (A_{bar(Bott)} / A_{s\ REQ'D})$ x 12 =	7.547 in	
Check that the section is not over-reinforced, AASHTO 5.7.3.3.1: $c / d = a / \beta_1 / d$ =	0.188	**OK! < 0.42**

Negative Moment

$A_{s\ PRV'D}$ =	0.676 in^2/ft	
Effective Depth, $d = T_S - C_{Vt} - d_{bar}/2$ =	5.688 in	
$M_{U(D+LL+I)} = M_{U(D)} + M_{U(LL+I)}$ =	7.39 k-ft/ft	
$\rho_{PRV'D}$ =	0.00991	
$a = A_{s(Top)} f_y / 0.85 f'_c b$ =	0.884 in	
$\phi M_n = \phi A_s f_y (d - a/2)$ =	15.97 k-ft/ft	**OK! > Mu**
$R_N = M_U / (\phi b d^2)$ =	0.25 ksi	
$\rho_{REQ'D} = .85 (f'_c / f_y) \{1 - [1 - 2 R_N / (.85 f'_c)]^{.5}\}$ =	0.00438	
$A_{s\ REQ'D} = \rho_{REQ'D} b d$ =	0.299 in^2/ft	
Req'd Bar Spacing $= (A_{bar(Top)} / A_{s\ REQ'D})$ x 12 =	12.447 in	
Check that the section is not over-reinforced, AASHTO 5.7.3.3.1: $c / d = a / \beta_1 / d$ =	0.188	**OK! < 0.42**

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LRFD Design - One Course Construction - Summary of Design Criteria and Output — LEFT OVERHANG

Longitudinal Distribution Steel

Bottom

AASHTO 9.7.3.2 - For primary reinforcement perpendicular to traffic

% Distribution Steel = 220 / $(S_{eff})^{.5}$ < 67% = 76.98% **Use 67.00%**

Area of Steel, $A_{s(Dist)}$ = % x $A_{s,eff.(Bott)}$ = 0.453 in^2

Required Spacing = (A_{bar} / A_{sDist}) x 12 for #5 bars = 8.21 in

Top ~~**Use #5's @ 8"**~~ **Use #5 @ 13" per ODOT BDM Fig. 302.2.2-1**

AASHTO 5.10.8 & 5.10.3.2

Area of Steel, A_s >=1.30bh / 2(b+h)f_y = 0.054 in^2/ft

0.11 <= A_s <= 0.60 0.110 in^2

Required Spacing = (A_{bar} / A_{sDist}) x 12 for #4 bars = 21.82 in

Maximum Spacing = 3.0 x T_S < 18 in. = 18.00 in

~~**Use #4's @ 8" Min. for Crack Control**~~ **Use #4 @ 12.5" per ODOT BDM Fig. 302.2.2-1**

4) Minimum Reinforcement

(AASHTO 5.4.2.6,& 5.7.3.3.2)

For Normal Weight Concrete $f_r = 0.37(f'_c)^{.5}$ = 0.78 ksi

$I_g = (1/12)*bh^3$ = 421.9 in^4

$y_t = h/2$ = 3.75 in

1.2 $M_{cr} = f_r (I_g / y_t)$ = 8.83 k-ft/ft

Positive Moment

ϕM_n must be greater than 1.2M_{cr}: **OK!**

If 1.2 M_{cr} > M_u, then check that ϕM_n need not be greater than 4/3 M_u

4/3$M_{U(D+LL+I)}$ = 15.87 k-ft/ft **Check Not Needed !!**

Negative Moment

ϕM_n must be greater than 1.2M_{cr}: **OK!**

If 1.2 M_{cr} > M_u, then check that ϕM_n need not be greater than 4/3 M_u

4/3$M_{U(D+LL+I)}$ = 9.85 k-ft/ft **Check Not Needed !!**

5) Check Crack Control

(AASHTO 5.7.3.4)

Exposure Factor, γ_e = 0.75 k/in

Compute Service Load Stress:

Note: Based on previous calculations, the section does not exceed the cracked moment per AASHTO provisions and therefore is assumed uncracked.

Positive Moment

$M_{D+LL+I} = M_D + M_{LL+I}$ = 6.96 k-ft/ft

k =$[(\rho n)^2+2 \rho n]^{.5} - \rho n$ = 0.312 j = 1 - k/3 = 0.896

C = T = M_{D+LL+I} / (j d) = 16.38 k

$f_s = T / A_s$ = 24.22 ksi

$f_c = 2 C / (b k d)$ = 1.539 ksi

Negative Moment

$M_{D+LL+I} = M_D + M_{LL+I}$ = 4.38 k-ft/ft

k =$[(\rho n)^2 + 2 \rho n]^{.5} - \rho n$ = 0.312 j = 1 - k/3 = 0.896

C = T = M_{D+LL+I} / (j d) = 10.31 k

$f_s = T / A_s$ = 15.24 ksi

$f_c = 2 C / (b k d)$ = 0.968 ksi

Determine Maximum Allowable Spacing:

Positive Moment

Depth From Extreme Tension Fiber to Closest Bar Center, $d_c = C_{Vb} + d_{bar}/2$ = 1.813 in

Strain Ratio, $\beta_s = 1 + d_c / [0.7 * (h - d_c)]$ = 1.455 Where h is equal to the slab thick. minus wearing surface, $T_s - T_w$

Spacing limit, $s_{max} = (700 * \gamma_e / \beta_s f_s) - 2 * d_c$ = 11.27 in

Actual Spacing = 5.50 in **OK! < Max spa**

Negative Moment

Depth From Extreme Tension Fiber to Closest Bar Center, $d_c = C_{Vt} + d_{bar} / 2$ = 2.81 in

Strain Ratio, $\beta_s = 1 + d_c / [0.7 * (h - d_c)]$ = 1.706 Where h is equal to the slab thickness, T_s

Spacing limit, $s_{max} = (700 * \gamma_e / \beta_s f_s) - 2 * d_c$ = 14.57 in

Actual Spacing = 5.50 in **OK! < Max spa**

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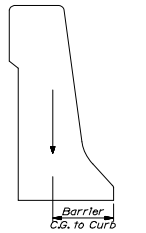
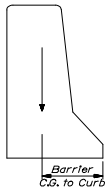
OVERHANG DESIGN: AASHTO A13.4

Concrete Barrier Properties

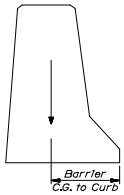
DOT or Project Specific Barrier Type	ODOT - 42in.
Barrier Area =	4.059 ft^2
Barrier CG to Curb Line, B_{CG} =	0.903 ft
Barrier Height, B_H =	3.500 ft
Barrier Weight, B_W =	0.609 klf
Equivalent Width of Barrier as a Uniform Section	
Modified "w" Value To Be Used? =	ODOT - 42in.
w = Barrier Area / Barrier Height =	1.160 ft



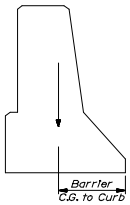
NJTA, NJDOT & WVDOT Barrier Type



INDOT, KYTC & MDOT Barrier Type



IDOT Barrier Type



ODOT Barrier Type

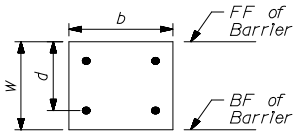
Step 1 -Determine Moment Capacity of Barrier

AASHTO A13.3.1

A) Capacity of Barrier about an Axis Parallel to the Longitudinal Axis of the Bridge:

Consider 1' Section of Barrier

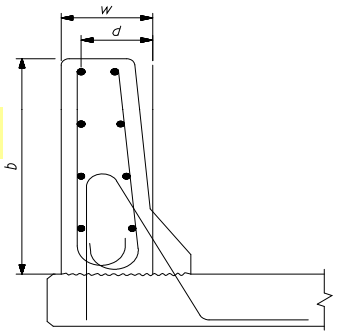
f'_c =	4.5 ksi	
Yield Stress of Reinforcement, f_y =	60.0 ksi	
Vertical Reinforcement	#5 bar spa. @	11.00 in
A_s =	0.338 in^2/ft	
Dia. =	0.625 in	
Cover, C_V =	2.0 in	
b =	12.00 in	
d = w - Cv - Dia. / 2 =	11.604	
$M_c = \phi M_n = \phi\ A_s f_y (d - a/2) / b$ =	19.2 k-ft/ft	



B) Capacity of Barrier about its Vertical Axis:

Consider Equivalent Width Barrier

Top Half Reinforcement	1	#6 bars
Bottom Half Reinforcement	6	#5 bars
A_s =	2.300 in^2	
Diameter of Vertical Reinforcement =	0.625 in	
Average Diameter of Longitudinal Reinforcement =	0.688 in	
Cover, C_V =	2.0 in	
(In this instance is equal to height of barrier) b =	42.00 in	
d = w - Cv - Dia_V - Dia_L / 2 =	10.948	
$M_w = \phi M_n = \phi\ A_s f_y (d - a/2)$ =	121.0 k-ft	



Step 2 - Determine Barrier Resistance to Transverse Loads, R_w:

A) For Impacts Within a Wall Segment: AASHTO A13.3.1

$R_w = (2 / (2L_c - L_t))(8\ M_b + 8\ M_w + M_c\ L_c^2 / H)$

Where:	M_b =	0.0 k-ft	*(additional flexural resistance at top of barrier, if any)*
	Height of Barrier, H =	3.500 ft	
	Test Level =	TL-2	
	L_t =	4.0 ft	Table A13.2-1 for appropriate Test Level
	Distance Between Vertical Open Joints in Barrier =	25.000 ft	
	$L_c = L_t / 2 + [(L_t / 2)^2 + 8\ H\ (M_b + M_w) / M_c]^{0.5}$ =	15.4 ft	**Use 15.4 ft**
	R_w =	169.55 kips	Max. 1.25 x F_t from Table A13.2-1
	USE R_w =	33.75 kips	**Does Not Govern!**

AASHTO A13.4.2

For Design Case 1, the deck overhang may be designed to provide a flexural resistance, M_s (k-ft/ft) which, acting coincident with the tensile force P_T (k/ft) specified herein, exceeds M_c of the parapet **at its base**.

Design Axial Tensile Force, $P_T = R_w / (L_c + 2\ H)$ = 1.51 k/ft

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B) For Impacts at End of Wall or at Joint: AASHTO A13.3.1

$R_w = (2 / (2L_c - L_t))(M_b + M_w + M_c L_c^2 / H)$

Where: M_b =	0.0 k-ft	*(additional flexural resistance at top of barrier, if any)*	
Height of Barrier, H =	3.500 ft		
L_t =	4.0 ft	Table A13.2-1 for appropriate Test Level	
Distance Between Vertical Open Joints in Barrier =	25.000 ft		
$L_c = L_t / 2 + [((L_t / 2)^2 + H (M_b + M_w) / M_c)]^{0.5}$ =	7.1 ft	**Use 7.1 ft**	
R_w =	78.07 kips	Max. 1.25 x F_t from Table A13.2-1	
USE R_w =	33.75 kips	**Does Not Govern!**	

AASHTO A13.4.2

For Design Case 1, the deck overhang may be designed to provide a flexural resistance, M_s (k-ft/ft) which, acting coincident with the tensile force P_T (k/ft) specified herein, exceeds M_c of the parapet **at its base**.

Design Axial Tensile Force, $P_T = R_w / (L_c + 2 H)$ = 2.39 k/ft

Step 3 - Evaluate Various Extreme Event and Strength Limit State Cases:

Part 1) Design Case 1, at Base of Barrier : Horizontal Vehicular Collision Load - Extreme Event Limit State

Dead Load Moments

Uniform Load of Slab, $W_{DC1} = (T_{O\text{-}Min} + T_O)/2 / 12 * \gamma_{conc}$ = 0.11 ksf

Load Factor, γ_{pDC} = 1.25 For M_{DC} *AASHTO Table 3.4.1-1*

Uniform Load of Overlay, W_{DC2}, and Overlay Moment, M_{DC2} not present

Load Modifier, η_i = 1.0 For All Loads

Moments

Resistance Factor, ϕ = 1.0 For Extreme Event Limit State

Slab Moment, $M_{DC1} = W_{DC1} (O - C)^2 / 2$ =	0.12 k-ft/ft	
Barrier Moment, $M_{DC3} = B_W B_{CG}$ =	0.55 k-ft/ft	*Note: In an Extreme Event Limit State load combination, wind on structure is not included, and therefore is not included as a load on the noise barrier/fence.*
Noise Barrier/Fence Moment, $M_{DC4} = N_W N_{CG}$ =	0.00 k-ft/ft	
Total Dead Load Service Moment =	0.67 k-ft/ft	
Total DL Factored $M_{U(D)} = \eta_i \gamma_{pDC} (M_{DC1} + M_{DC3} + M_{DC4})$ =	0.84 k-ft/ft	
Total Design Factored Moment, $M_u = M_c + M_{U(D)}$ =	20.08 k-ft/ft	

Check Moment Capacity of Reinforcement

For a section under moment and axial tension, P_T, the nominal resistance, M_n, may be calculated as: $\phi M_n = \phi[T(d - a/2) - P_T(d/2 - a/2)]$

$A_{s(Top)}$ =	0.676 in^2/ft	$A_{s(Add'l)}$ =	0.169 in^2/ft
Avg. Dia., $d_{avg} = (A_{s(Top)} Dia_{Top} + A_{s(Add'l)} Dia_{Add'l}) / A_{sTotal}$ =	0.625 in	b =	12.00 in
Effective Depth, $d = (T_{O\text{-}Min} + T_O)/2 - C_{Vt} - d_{avg}/2$ =	5.688 in	$\rho_{PRV'D}$ =	0.01239
$T = (A_{s(Top)} + A_{s(Add'l)}) f_y$ =	50.73 k/ft		
Compression in Concrete, $C = T - P_T$ =	48.33 k/ft	Where P_T is the Design Axial Tensile Force from Step 2 Above	
$a = C / b \beta_1 f'_c$ =	1.085 in		
ϕM_n =	21.29 k-ft/ft	**OK! > Mu**	
Check that the section is not over-reinforced, AASHTO 5.7.3.3.1: $c / d = a / \beta_1 / d$ =	0.231	**OK! < 0.42**	

Part 2) Design Case 1, at Critical Section, Overhang Side : Horizontal Vehicular Collision Load - Extreme Event Limit State

Assume the distribution of collision loads increases by 0.8X as the analysis point moves X ft away from the barrier. 0.8 corresponds to a distribution angle of 22° for the increase of the force distribution from the barrier face similar to the AASHTO Standard LFD Specifications.

Design Moment due to Collision Acting at Critical Section: $M_{CS} = M_c L_c / (L_c + 0.8X)$

Where: M_c =	19.2 k-ft/ft
L_c =	15.4 ft
$X = C - E$ =	1.047 ft
M_{CS} =	18.26 k-ft/ft

Design Axial Tensile Force Acting at Critical Section:

$P_{CS} = R_w / (L_c + 2 H + 0.8X)$ = 1.45 k/ft

Dead Load Moments

Uniform Load of Slab, $W_{DC1} = (T_{O\text{-}Min} + T_O)/2 / 12 * \gamma_{conc}$ = 0.11 ksf

Load Factor, γ_{pDC} = 1.25 For M_{DC} *AASHTO Table 3.4.1-1*

Uniform Load of Overlay, $W_{DC2} = T_L / 12 * \gamma_{conc}$ = 0.00 ksf

Load Factor, γ_{pDW} = 1.50 For M_{DW}

Uniform Load of Sidewalk, $W_{DC5} = T_{SW}/12 * \gamma_{conc}$ = 0.00 ksf

Load Modifier, η_i = 1.0 For All Loads

Moments

Resistance Factor, ϕ = 1.0 For Extreme Event Limit State

Slab Moment, $M_{DC1} = W_{DC1} (O - E)^2 / 2$ =	0.34 k-ft/ft
Overlay Moment, $M_{DC2} = W_{DC2} (C - E)^2 / 2$ =	0.00 k-ft/ft
Barrier Moment, $M_{DC3} = B_W (B_{CG} + C - E)$ =	1.19 k-ft/ft
Noise Barrier/Fence Moment, $M_{DC4} = N_W (N_{CG} + C - E)$ =	0.00 k-ft/ft
Sidewalk Moment, $M_{DC5} = W_{DC5} (C - E)^2 / 2$ =	0.00 k-ft/ft
FWS Moment, $M_{DW} = W_{DW} (C - E)^2 / 2$ =	0.03 k-ft/ft
Total Dead Load Service Moment =	1.56 k-ft/ft
Total DL Fact. $M_{U(D)} = \eta_i(\gamma_{pDC} (M_{DC1} + M_{DC2} + M_{DC3} + M_{DC4} + M_{DC5}) + \gamma_{pDW} M_{DW})$ =	1.96 k-ft/ft
Total Design Factored Moment, $M_u = M_{CS} + M_{U(D)}$ =	20.22 k-ft/ft

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For a section under moment and axial tension, P_{CS}, the nominal resistance, M_n, may be calculated as: $\phi M_n = \phi[T(d - a/2) - P_{CS}(d/2 - a/2)]$

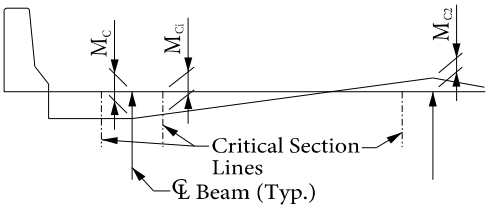
$A_{s(Top)}$ =	0.676 in^2/ft	$A_{s(Add'l)}$ =	0.169 in^2/ft	
Avg. Dia., $d_{avg} = (A_{s(Top)}\ Dia_{Top} + A_{s(Add'l)}\ Dia_{Add'l}) / A_{sTotal}$ =	0.625 in	b =	12.00 in	
Effective Depth, $d = T_O - C_{Vt} - d_{avg}/2$ =	5.688 in	$\rho_{PRV'D}$ =	0.01239	
$T = (A_{s(Top)} + A_{s(Add'l)})\ f_y$ =	50.73 k/ft			
Compression in Concrete, $C = T - P_{CS}$ =	49.28 k/ft	Where P_{CS} is the Design Axial Tensile Force from above		
$a = C / b\ \beta_1\ f'_c$ =	1.106 in			
ϕM_n =	21.43 k-ft/ft	**OK! > Mu**		
Check that the section is not over-reinforced, AASHTO 5.7.3.3.1: $c / d = a / \beta_1 / d$ =	0.236	**OK! < 0.42**		

Part 3) Design Case 1, at Critical Section, Interior Span Side : Horizontal Vehicular Collision Load - Extreme Event Limit State

Assume the collision moment at the first interior beam, $M_{c2} = (O / B) * M_c$

Interpolate to find the collision moment at the interior side, $M_{ci} = M_c - I * (M_c + M_{c2}) / B$

$A = 1 - (\text{Min. of bf / 3 or 1.25 ft}) * (1 + O / B) / B$ =	0.798	
Design Moment due to Collision Acting at Critical Section: $M_{CS} = M_{ci}\ L_c / (L_c + 0.8X)$		
M_{c2} =	8.3 k-ft/ft	
M_{ci} =	15.4 k-ft/ft	
L_c =	15.4 ft	
$X = C + E$ =	3.547 ft	
M_{CS} =	12.97 k-ft/ft	
Design Axial Tensile Force Acting at Critical Section:		
$P_{CS} = R_w / (L_c + 2\ H + 0.8X)$ =	1.34 k/ft	



Dead Load Moments

Uniform Load of OH Slab, $W_{DC1} = (T_{O\text{-}Min} + T_O)/2/\ 12 * \gamma_{conc}$ =	0.11 ksf	Load Factor, γ_{pDC} =	1.25	For M_{DC}	*AASHTO Table 3.4.1-1*
Uniform Load of Overlay, $W_{DC2} = T_L / 12 * \gamma_{conc}$ =	0.00 ksf	Load Factor, γ_{pDW} =	1.5	For M_{DW}	
Uniform Load of Interior Slab, $W_{DC1i} = T_{Si} / 12 * \gamma_{conc}$ =	0.11 ksf	Load Modifier, η_i =	1.0	For All Loads	
Uniform Load of Sidewalk, $W_{DC5} = T_{SW} / 12 * \gamma_{conc}$ =	0.00 ksf	Resistance Factor, ϕ =	1.0	For Extreme Event Limit State	
Uniform Load of Interior Sidewalk, $W_{DC5i} = T_{SWi} / 12 * \gamma_{conc}$ =	0.00 ksf				

Moments

Exterior Slab Moment, $M_{DC1} = A\ W_{DC1}\ (O)^2 / 2$ =	0.61 k-ft/ft
Exterior Overlay Moment, $M_{DC2} = A\ W_{DC2}\ (C)^2 / 2$ =	0.00 k-ft/ft
Interior Slab Moment, $M_{DC1i} = 0.4W_{DC1i}\ (B)(E) - W_{DC1i}\ (E)^2 / 2$ =	-0.39 k-ft/ft
Interior Overlay Moment, $M_{DC2i} = 0.4W_{DC2}\ (B)(E) - W_{DC2}\ (E)^2 / 2$ =	0.00 k-ft/ft
Barrier Moment, $M_{DC3} = A\ B_W\ (B_{CG} + C)$ =	1.55 k-ft/ft
Noise Barrier/Fence Moment, $M_{DC4} = A\ N_W\ (N_{CG} + C)$ =	0.00 k-ft/ft
Sidewalk Moment, $M_{DC5} = A\ W_{DC5}\ (C)^2 / 2$ =	0.00 k-ft/ft
Interior Sidewalk Moment, $M_{DC5i} = 0.5\ W_{DC5i}\ (L_{SW} - C)(I) - W_{DC5i}\ (I)^2 / 2$ =	0.00 k-ft/ft
FWS Moment, $M_{DW} = A\ W_{DW}\ (C)^2 / 2$ =	0.13 k-ft/ft
Interior FWS Moment, $M_{DWi} = 0.4M_{DW}\ (B)(E) - M_{DW}\ (E)^2 / 2$ =	-0.22 k-ft/ft
Total Dead Load Service Moment =	1.69 k-ft/ft
Total Fact. $M_{U(D)} = \eta_i(\gamma_{pDC}(M_{DC1}+M_{DC2}+M_{DC1i}+M_{DC2i}+M_{DC3}+M_{DC4}+M_{DC5}+M_{DC5i})+\gamma_{pDW}(M_{DW}+M_{DWi})$ =	2.09 k-ft/ft
Total Design Factored Moment, $M_u = M_{CS} + M_{U(D)}$ =	15.05 k-ft/ft

For a section under moment and axial tension, P_{CS}, the nominal resistance, M_n, may be calculated as: $\phi M_n = \phi[T(d - a/2) - P_{CS}(d/2 - a/2)]$

$A_{s(Top)}$ =	0.676 in^2/ft	$A_{s(Add'l)}$ =	0.169 in^2/ft	
Avg. Dia., $d_{avg} = (A_{s(Top)}\ Dia_{Top} + A_{s(Add'l)}\ Dia_{Add'l}) / A_{sTotal}$ =	0.625 in	b =	12.00 in	
Effective Depth, $d = T_{Si} - C_{Vt} - d_{avg}/2$ =	5.688 in	$\rho_{PRV'D}$ =	0.01239	
$T = (A_{s(Top)} + A_{s(Add'l)})\ f_y$ =	50.73 k/ft			
Compression in Concrete, $C = T - P_{CS}$ =	49.39 k/ft	Where P_{CS} is the Design Axial Tensile Force from above		
$a = C / b\ \beta_1\ f'_c$ =	1.109 in			
ϕM_n =	21.44 k-ft/ft	**OK! > Mu**		
Check that the section is not over-reinforced, AASHTO 5.7.3.3.1: $c / d = a / \beta_1 / d$ =	0.236	**OK! < 0.42**		

Part X) Design Case 2: Vertical Vehicular Collision Load - Extreme Event Limit State

This case does not control design for concrete barriers and was therefore not checked.

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Part 4) Design Case 3, at Critical Section, Overhang Side : The loads that occupy the Overhang - Strength Limit State I

Note: For the noise barrier/fence, wind load is not included under this load combination.

Load Factor, γ_{pDC} =	1.25	For M_{DC}	*AASHTO Table 3.4.1-1*
Load Factor, γ_{pDW} =	1.5	For M_{DW}	
Load Factor, γ_{LL} =	1.75	For M_{LL}	
Load Modifier, η_i =	1.00	For All Loads	
Resistance Factor, ϕ =	0.9	For Strength Limit State	

Dead Load Moments

Uniform Load of Slab, $W_{DC1} = (T_{O\text{-}Min} + T_O)/2 / 12 * \gamma_{conc}$ =	0.11 ksf
Uniform Load of Overlay, $W_{DC2} = T_L / 12 * \gamma_{conc}$ =	0.00 ksf
Uniform Load of Sidewalk, $W_{DC5} = T_{sw} / 12 * \gamma_{conc}$ =	0.00 ksf
Moments	
Slab Moment, $M_{DC1} = W_{DC1} (O - E)^2 / 2$ =	0.34 k-ft/ft
Overlay Moment, $M_{DC2} = W_{DC2} (C - E)^2 / 2$ =	0.00 k-ft/ft
Barrier Moment, $M_{DC3} = B_W (B_{CG} + C - E)$ =	1.19 k-ft/ft
Noise Barrier/Fence Moment, $M_{DC4} = N_W (N_{CG} + C - E)$ =	0.00 k-ft/ft
Sidewalk Moment, $M_{DC5} = W_{DC5} (C - E)^2 / 2$ =	0.00 k-ft/ft
FWS Moment, $M_{DW} = W_{DW} (C - E)^2 / 2$ =	0.03 k-ft/ft
Total Dead Load Service Moment =	1.56 k-ft/ft
Total DL Fact. $M_{U(D)} = \eta_i(\gamma_{pDC}(M_{DC1} + M_{DC2} + M_{DC3} + M_{DC4} + M_{DC5}) + \gamma_{pDW} M_{DW})$ =	1.96 k-ft/ft

Live Load Moment

Multiple Presence Factor, m =	0.65	AASHTO Table 3.6.1.1.2-1 for 2 Lanes Loaded
Dynamic Load Allowance (Impact), IM =	1.33	AASHTO 3.6.2.1
X = C - E - 1ft =	0.047 ft	
Equivalent Strip Width for Live Load, W = 45 + 10 X =	45.468 in	AASHTO Table 4.6.2.1.3-1 (X is in feet)
Wheel Load, P =	16.0 kips	
M_{LL+I} = m (IM) P X / W =	0.17 k-ft/ft	
Factored LL + I Moment, $M_{U(LL+I)} = \eta_i \gamma_{LL} M_{(LL+I)}$ =	0.30 k-ft/ft	

Check Moment Capacity of Reinforcement

$A_{s(Top)}$ =	0.676 in^2/ft	$A_{s(Add'l)}$ =	0.169 in^2/ft
Avg. Dia., $d_{avg} = (A_{s(Top)} Dia_{Top} + A_{s(Add'l)} Dia_{Add'l}) / A_{sTotal}$ =	0.625 in	b =	12.00 in
Effective Depth, $d = T_O - C_{Vt} - d_{avg}/2$ =	5.688 in	$\rho_{PRV'D}$ =	0.01239
$M_{U(D+LL+I)} = M_{U(D)} + M_{U(LL+I)}$ =	2.26 k-ft/ft		
$a = A_{sTotal} f_y / 0.85 f'_c b$ =	1.105 in		
$\phi M_n = \phi A_{sTotal} f_y (d - a/2)$ =	19.54 k-ft/ft	**OK! > Mu**	

Check Crack Control

(AASHTO 5.7.3.4)

Total Service $M_{D+LL+I} = M_D + M_{LL+I}$ =	1.74 k-ft/ft		
$k = [(\rho n)^2 + 2 \rho n]^{.5} - \rho n$ =	0.341	j = 1 - 1/3*k =	0.886
$C = T = M_{D+LL+I} / (j d)$ =	4.1 k		
$f_s = T / A_{sTotal}$ =	4.89 ksi		
$f_c = 2 C / (b k d)$ =	0.35 ksi		
Exposure Factor, γ_e =	0.75 k/in		
Depth From Extreme Tension Fiber to Closest Bar Center, $d_c = C_{Vt} + d_{avg}/2$ =	2.81 in		
Strain Ratio, $\beta_s = 1 + d_c / [0.7 * (h - d_c)]$ =	1.706	Where h is equal to the maximum overhang slab thickness T_O	
Spacing Limit, $s_{max} = (700 * \gamma_e / \beta_s f_s) - 2 * d_c$ =	57.32 in		
Actual Spacing =	5.50 in	**OK! < Max spa**	

Part 5) Design Case 3, at Critical Section, Overhang Side : The loads that occupy the Overhang - Strength Limit State V

Note: For this load combination, wind load is included on the noise barrier/fence.

Load Factor, γ_{LL} =	1.35	For M_{LL}	*AASHTO Table 3.4.1-1*
Load Factor, γ_{WS} =	0.40	For M_{WS}	

Dead Load Moments

Total DL Fact. $M_{U(D)} = \eta_i(\gamma_{pDC}(M_{DC1} + M_{DC2} + M_{DC3} + M_{DC4} + M_{DC5}) + \gamma_{pDW} M_{DW})$ =	1.96 k-ft/ft	Same as Part 4)

Live Load Moment

Factored LL + I Moment, $M_{U(LL+I)} = \eta_i \gamma_{LL} M_{(LL+I)}$ =	0.23 k-ft/ft	Same as Part 4) above, except with new factor

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Noise Barrier/Fence Wind Moment

$M_{WS} = P_W (N_H - B_H) ((N_H - B_H)/2 + B_H + T_L + T_O/2) =$	0.00 k-ft/ft	
Factored Wind Moment, $M_{U(WS)} = \eta_i \gamma_{WS} M_{WS} =$	0.00 k-ft/ft	
Total Fact. Moment = $M_{U(D)} + M_{U(LL+I)} + M_{U(WS)} =$	2.19 k-ft/ft	

Check Moment Capacity of Reinforcement

$\phi M_n =$	19.54 k-ft/ft	**OK! > Mu**

Check Crack Control

Total Service Moment = $M_D + M_{LL+I} + M_{WS} =$	1.74 k-ft/ft	
$f_s = M / A_{sTotal}\ j\ d =$	4.89 ksi	
Exposure Factor, $\gamma_e =$	0.75 k/in	
Depth From Extreme Tension Fiber to Closest Bar Center, $d_c = C_{Vt} + d_{avg}/2 =$	2.81 in	
Strain Ratio, $\beta_s = 1 + d_c / [0.7 * (h-d_c)]$	1.706	Where h is equal to the max overhang slab thickness T_O
Spacing Limit, $s_{max} = (700 * \gamma_e / \beta_s f_s) - 2 * d_c =$	57.32 in	
Actual Spacing =	5.50 in	**OK! < Max spa**

Part 6) Design Case 3, at Critical Section, Interior Span Side : The loads that occupy the Overhang - Strength Limit State I

Note: For the noise barrier/fence, wind load is not included under this load combination.

Load Factor, $\gamma_{pDC} =$	1.25	For M_{DC}	*AASHTO Table 3.4.1-1*
Load Factor, $\gamma_{pDW} =$	1.5	For M_{DW}	
Load Factor, $\gamma_{LL} =$	1.75	For M_{LL}	
Load Modifier, $\eta_i =$	1.05	For All Loads	
Resistance Factor, $\phi =$	0.9	For Strength Limit State	

Dead Load Moments

Uniform Load of OH Slab, $W_{DC1} = (T_{O\text{-}Min} + T_O) / 2 / 12 * \gamma_{conc} =$	0.11 ksf	
Uniform Load of Overlay, $W_{DC2} = T_L / 12 * \gamma_{conc} =$	0.00 ksf	
Uniform Load of Interior Slab, $W_{DC1\text{-}i} = T_S / 12 * \gamma_{conc} =$	0.11 ksf	
Uniform Load of Sidewalk, $W_{DC5} = T_{SW} / 12 * \gamma_{conc} =$	0.00 ksf	
Uniform Load of Interior Sidewalk, $W_{DC5i} = T_{SWi} / 12 * \gamma_{conc} =$	0.00 ksf	
Moments		
Slab Moment, $M_{DC1} = A\ W_{DC1}\ (O)^2 / 2 =$	0.61 k-ft/ft	
Overlay Moment, $M_{DC2} = A\ W_{DC2}\ (C)^2 / 2 =$	0.00 k-ft/ft	
Interior Slab Moment, $M_{DC1i} = 0.4 W_{DC1i}\ (B)(E) - W_{DC1i}\ (E)^2 / 2 =$	-0.39 k-ft/ft	
Interior Overlay Moment, $M_{DC2\text{-}i} = 0.4 W_{DC2}\ (B)(E) - W_{DC2}\ (E)_2 / 2 =$	0.00 k-ft/ft	
Barrier Moment, $M_{DC3} = A\ B_W\ (B_{CG} + C) =$	1.55 k-ft/ft	
Noise Barrier/Fence Moment, $M_{DC4} = A\ N_W\ (N_{CG} + C) =$	0.00 k-ft/ft	
Sidewalk Moment, $M_{DC5} = A\ W_{DC5}\ (C)^2 / 2 =$	0.00 k-ft/ft	
Interior Sidewalk Moment, $M_{DC5i} = 0.5\ W_{DC5i}\ (L_{SW} - C)(I) - W_{DC5i}\ (I)^2 / 2 =$	0.00 k-ft/ft	
FWS Moment, $M_{DW} = A\ W_{DW}\ (C)^2 / 2 =$	0.13 k-ft/ft	
FWS Moment, $M_{DWi} = 0.4 W_{DW}\ (B)(E) - W_{DW}\ (E)^2 / 2 =$	-0.22 k-ft/ft	
Total Dead Load Service Moment =	1.69 k-ft/ft	
Total Fact. $M_{U(D)} = \eta_i(\gamma_{pDC}(M_{DC1}+M_{DC2}+M_{DC1i}+M_{DC2i}+M_{DC3}+M_{DC4}+M_{DC5}+M_{DC5i})+\gamma_{pDW}(M_{DW}+M_{DWi}) =$	2.19 k-ft/ft	

Live Load Moment

Multiple Presence Factor, m =	1.20	AASHTO Table 3.6.1.1.2-1 for 1 Lane Loaded
Dynamic Load Allowance (Impact), IM =	1.33	AASHTO 3.6.2.1
If C > 2' then M_{LL} =	[P x ((C - 2) + E) - (P x (B + (C - 2)) + P x (B - (6 - (C - 2)))) / B x E] + P x (E - (2 - C))	
Else M_{LL} =	[-P x ((B - (2 - C)) + (B - (2 - C) - 6)) / B x E] + P x (E - (2 - C))	
$M_{LL} =$	21.74 k-ft	
Equivalent Strip Width for Live Load, W = 45 + 10 X =	74.500 in	AASHTO Table 4.6.2.1.3-1 (S is in feet)
Wheel Load, P =	16.0 kips	
$M_{LL+I} = m\ (IM)\ M_{LL} / W =$	5.59 k-ft/ft	
Factored LL + I Moment, $M_{U(LL+I)} = \eta_i \gamma_{LL} M_{(LL+I)} =$	10.27 k-ft/ft	

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Check Moment Capacity of Reinforcement

$A_{s(Top)}$ =	0.676 in^2/ft	$A_{s(Add'l)}$ =	0.169 in^2/ft	
Avg. Dia., $d_{avg} = (A_{s(Top)}\ Dia_{Top} + A_{s(Add'l)}\ Dia_{Add'l}) / A_{sTotal}$ =	0.625 in	b =	12.00 in	
Effective Depth, $d = T_S - C_{Vt} - d_{avg}/2$ =	5.688 in	$\rho_{PRV'D}$ =	0.01239	
$M_{U(D+LL+I)} = M_{U(D)} + M_{U(LL+I)}$ =	12.46 k-ft/ft			
$a = A_{sTotal}\ f_y / 0.85\ f'_c\ b$ =	1.105 in			
$\phi M_n = \phi\ A_{sTotal}\ f_y\ (d - a/2)$ =	19.54 k-ft/ft	**OK! > Mu**		

Check Crack Control

(AASHTO 5.7.3.4)

Total Service $M_{D+LL+I} = M_D + M_{LL+I}$ =	7.28 k-ft/ft		
$k = [(\rho\ n)^2 + 2\ \rho\ n]^{.5} - \rho\ n$ =	0.341	j = 1 - 1/3*k =	0.886
$C = T = M_{D+LL+I} / (j\ d)$ =	17.3 k		
$f_s = T / A_{sTotal}$ =	20.49 ksi		
$f_c = 2\ C / (b\ k\ d)$ =	1.49 ksi		
Exposure Factor, γ_e =	0.75 k/in		
Depth From Extreme Tension Fiber to Closest Bar Center, $d_c = C_{Vt} + d_{avg} / 2$ =	2.81 in		
Strain Ratio, $\beta_s = 1 + d_c / [0.7 * (h-d_c)]$ =	1.706	Where h is equal to the interior slab thickness T_S	
Spacing Limit, $s_{max} = (700 * \gamma_e / \beta_s f_s) - 2 * d_c$ =	9.39 in		
Actual Spacing =	5.50 in	**OK! < Max spa**	

Part 7) Design Case 3, at Critical Section, Interior Span Side : The loads that occupy the Overhang - Strength Limit State V

Note: For this load combination, wind load is included on the noise barrier/fence.

Load Factor, γ_{LL} =	1.35	For M_{LL}	*AASHTO Table 3.4.1-1*
Load Factor, γ_{WS} =	0.40	For M_{WS}	

Dead Load Moments

Total DL Fact. $M_{U(D)} = \eta_i(\gamma_{pDC}(M_{DC1} + M_{DC2} + M_{DC3} + M_{DC4} + M_{DC5} + M_{DC5i}) + \gamma_{pDW} M_{DW})$ =	2.19 k-ft/ft	Same as Part 6) above

Live Load Moment

Factored LL + I Moment, $M_{U(LL+I)} = \eta_i\ \gamma_{LL}\ M_{(LL+I)}$ =	7.92 k-ft/ft	Same as Part 6) above, except with new factor

Noise Barrier/Fence Wind Moment

$M_{WS} = A *\ P_W\ (N_H)\ ((N_H)/2 + B_H + T_L + T_S/2)$ =	0.00 k-ft/ft
Factored Wind Moment, $M_{U(WS)} = \eta_i\ \gamma_{WS}\ M_{WS}$ =	0.00 k-ft/ft
Total Fact. Moment = $M_{U(D)} + M_{U(LL+I)} + M_{U(WS)}$ =	10.11 k-ft/ft

Check Moment Capacity of Reinforcement

ϕM_n =	19.54 k-ft/ft	**OK! > Mu**

Check Crack Control

(AASHTO 5.7.3.4)

Total Service Moment = $M_D + M_{LL+I} + M_{WS}$ =	7.28 k-ft/ft	
$f_s = M / A_{sTotal}\ j\ d$ =	20.49 ksi	
Exposure Factor, γ_e =	0.75 k/in	
Depth From Extreme Tension Fiber to Closest Bar Center, $d_c = C_{Vt} + d_{avg} / 2$ =	2.81 in	
Strain Ratio, $\beta_s = 1 + d_c / [0.7 * (h - d_c)]$ =	1.706	Where h is equal to the interior slab thickness T_S
Spacing Limit, $s_{max} = (700 * \gamma_e / \beta_s f_s) - 2 * d_c$ =	9.39 in	
Actual Spacing =	5.50 in	**OK! < Max spa**

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LRFD Design - One Course Construction - Summary of Design Criteria and Output — LEFT OVERHANG

Part 8) Compute Additional Reinforcing Overhang Cutoff Length Requirement

Note: This design only necessary when additional reinforcing steel is present in the overhang.

A) Check Add'l Reinforcing Cut-Off for Design Case 1, at Critical Section, Interior Span Side : Horizontal Vehicular Collision Load - Extreme Event Limit State

Location of Critical Section with Add'l No Longer Required =	10.0 in	Distance from CL Exterior Beam
Assume the collision moment at the first interior beam, $M_{c2} = 0.4M_c$		
Interpolate to find the collision moment at the interior span side, $M_{ci} = A \times M_c$, A =	0.868	
Design Moment due to Collision Acting at Critical Section: $M_{CS} = M_{ci} L_c / (L_c + 0.8X)$		
M_{c2}=	7.7 k-ft/ft	
M_{cx}=	16.7 k-ft/ft	
L_c =	15.4 ft	
X = C + L =	3.130 ft	
M_{CS} =	14.37 k-ft/ft	
Design Axial Tensile Force acting at Critical Section:		
$P_{CS} = R_w / (L_c + 2H + 0.8X)$ =	3.78 k/ft	

Dead Load Moments

Uniform Load of OH Slab, $W_{DC1} = (T_{O\text{-}Min} + T_O) / 2 / 12 * \gamma_{conc}$ =	0.11 ksf	Load Factor, γ_{pDC} =	1.25	For M_{DC}	*AASHTO Table 3.4.1-1*
Uniform Load of Overlay, $W_{DC2} = T_L / 12 * \gamma_{conc}$ =	0.00 ksf	Load Factor, γ_{pDW} =	1.5	For M_{DW}	
Uniform Load of Interior Slab, $W_{DC1i} = T_S / 12 * \gamma_{conc}$ =	0.11 ksf	Load Modifier, η_i =	1.0	For All Loads	
Uniform Load of Sidewalk, $W_{DC5} = T_{SW} / 12 * \gamma_{conc}$ =	0.00 ksf	Resistance Factor, ϕ =	1.0	For Extreme Event Limit State	
Uniform Load of Interior Sidewalk, $W_{DC5i} = T_{SWi} / 12 * \gamma_{conc}$ =	0.00 ksf				

Moments

Exterior Slab Moment, $M_{DC1} = A\ W_{DC1}\ (O)^2 / 2$ =	0.66 k-ft/ft
Exterior Overlay Moment, $M_{DC2} = A\ W_{DC2}\ (C)^2 / 2$ =	0.00 k-ft/ft
Interior Slab Moment, $M_{DC1i} = 0.4W_{DC1i}\ (B)(L) - W_{DC1i}\ (L)^2 / 2$ =	-0.28 k-ft/ft
Interior Overlay Moment, $M_{DC2i} = 0.4W_{DC2}\ (B)(L) - W_{DC2}\ (L)_2 / 2$ =	0.00 k-ft/ft
Barrier Moment, $M_{DC3} = A\ B_W\ (B_{CG} + C)$ =	1.69 k-ft/ft
Noise Barrier/Fence Moment, $M_{DC4} = A\ N_W\ (N_{CG} + C)$ =	0.00 k-ft/ft
Sidewalk Moment, $M_{DC5} = A\ W_{DC5}\ (C)^2 / 2$ =	0.00 k-ft/ft
Interior Sidewalk Moment, $M_{DC5i} = 0.5\ W_{DC5i}\ (L_{SW} - C)(L) - W_{DC5i}\ (L)^2 / 2$ =	0.00 k-ft/ft
FWS Moment, $M_{DW} = A\ W_{DW}\ (C)^2 / 2$ =	0.14 k-ft/ft
FWS Moment, $M_{DWi} = 0.4M_{DW}\ (B)(L) - M_{DW}\ (L)^2 / 2$ =	-0.16 k-ft/ft
Total Dead Load Service Moment =	2.06 k-ft/ft
Total Fact. $M_{U(D)} = \eta_i(\gamma_{pDC}(M_{DC1}+M_{DC2}+M_{DC1i}+M_{DC2i}+M_{DC3}+M_{DC4}+M_{DC5}+M_{DC5i})+\gamma_{pDW}(M_{DW}+M_{DWi})$ =	2.57 k-ft/ft
Total Design Factored Moment, $M_u = M_{CS} + M_{U(D)}$ =	16.94 k-ft/ft

For a section under moment and axial tension, P_{CS}, the nominal resistance, M_n, may be calculated as: $\phi M_n = \phi[T(d - a/2) - P_{CS}(d/2 - a/2)]$

$A_{s(Top)}$ =	0.676 in^2/ft	$A_{s(Add'l)}$ =	0.000 in^2/ft
Avg. Dia., $d_{avg} = (A_{s(Top)}\ Dia_{Top} + A_{s(Add'l)}\ Dia_{Add'l}) / A_{sTotal}$ =	0.625 in	b =	12.00 in
Effective depth, $d = T_S + T_L - C_{Vt} - d_{avg}/2$ =	5.688 in	$\rho_{PRV'D}$ =	0.00991
$T = (A_{s(Top)} + A_{s(Add'l)})\ f_y$ =	40.58 k/ft		
Compression in concrete, $C = T - P_{CS}$ =	36.80 k/ft	Where P_{CS} is the Design Axial Tensile Force from above	
$a = C / b\ \beta_1\ f'_c$ =	0.826 in		
ϕM_n =	17.07 k-ft/ft	**OK! > Mu**	
Check that the section is not over-reinforced, AASHTO 5.7.3.3.1: $c / d = a / \beta_1 / d$ =	0.176	**OK! < 0.42**	

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B) Check Add'l Steel Cut-Off for Design Case 3, at Critical Section, Interior Span Side : The loads that occupy the Overhang - Strength I

Note: For the noise barrier/fence, wind load is not included under this load combination.

Load Factor, γ_{pDC} =	1.25	For M_{DC}	*AASHTO Table 3.4.1-1*
Load Factor, γ_{pDW} =	1.5	For M_{DW}	
Load Factor, γ_{LL} =	1.75	For M_{LL}	
Load Modifier, η_i =	1.05	For All Loads	
Resistance Factor, ϕ =	0.9	For Strength Limit State	

Dead Load Moments

Uniform Load of OH Slab, $W_{DC1} = (T_{O\text{-}Min} + T_O) / 2 / 12 * \gamma_{conc}$ =	0.11 ksf	
Uniform Load of Overlay, $W_{DC2} = T_L / 12 * \gamma_{conc}$ =	0.00 ksf	
Uniform Load of Interior Slab, $W_{DC1i} = T_S / 12 * \gamma_{conc}$ =	0.11 ksf	
Moments		
Total Dead Load Service Moment =	2.06 k-ft/ft	
Total Fact. $M_{U(D)} = \eta_i(\gamma_{pDC}(M_{DC1}+M_{DC2}+M_{DC1i}+M_{DC2i}+M_{DC3}+M_{DC4}+M_{DC5}+M_{DC5i})+\gamma_{pDW}(M_{DW}+M_{DWi})$ =	2.70 k-ft/ft	

Live Load Moment

Multiple Presence Factor, m =	1.20	AASHTO Table 3.6.1.1.2-1 for 1 Lane Loaded
Dynamic Load Allowance (Impact), IM =	1.33	AASHTO 3.6.2.1
M_{LL} =	17.66 k-ft	
Equivalent Strip Width for Live Load, W = 48 + 3 S =	74.500 in	AASHTO Table 4.6.2.1.3-1 (S is in feet)
Wheel Load, P =	16.0 kips	
$M_{LL+I} = m\ (IM)\ M_{LL} / W$ =	4.54 k-ft/ft	
Factored LL + I Moment, $M_{U(LL+I)} = \eta_i \gamma_{LL} M_{(LL+I)}$ =	8.34 k-ft/ft	

Check Moment Capacity of Reinforcement

$A_{s(Top)}$ =	0.676 in^2/ft	$A_{s(Add'l)}$ =	0.000 in^2/ft
Avg. Dia., $d_{avg} = (A_{s(Top)}\ Dia_{Top} + A_{s(Add'l)}\ Dia_{Add'l}) / A_{sTotal}$ =	0.625 in	b =	12.00 in
Effective depth, $d = T_S + T_L - C_{Vt} - d_{avg}/2$ =	5.688 in	$\rho_{PRV'D}$ =	0.00991
$M_{U(D+LL+I)} = M_{U(D)} + M_{U(LL+I)}$ =	11.04 k-ft/ft		
$a = A_{sTotal}\ f_y / 0.85\ f'_c\ b$ =	0.884 in		
$\phi M_n = \phi\ A_{sTotal}\ f_y\ (d - a/2)$ =	15.97 k-ft/ft	**OK! > Mu**	

Check Crack Control

(AASHTO 5.7.3.4)

Total Service $M_{D+LL+I} = M_D + M_{LL+I}$ =	6.60 k-ft/ft		
$k = [(\rho\ n)^2 + 2\ \rho\ n]^{.5} - \rho\ n$ =	0.312	j = 1 - 1/3*k =	0.896
$C = T = M_{D+LL+I} / (j\ d)$ =	15.5 k		
$f_s = T / A_{sTotal}$ =	22.98 ksi		
$f_c = 2\ C / (b\ k\ d)$ =	1.46 ksi		
Exposure Factor, γ_e =	0.75 k/in		
Depth From Extreme Tension Fiber to Closest Bar Center, $d_c = C_{Vt} + d_{avg}/2$ =	2.81 in		
Strain Ratio, $\beta_s = 1 + d_c / [0.7 * (h\text{-}d_c)]$ =	1.706	Where h is equal to the interior slab thickness T_S	
Spacing Limit, $s_{max} = (700 * \gamma_e / \beta_s f_s) - 2 * d_c$ =	7.76 in		
Actual Spacing =	5.50 in	**OK! < Max spa**	

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C) Check Add'l Steel Cut-Off for Design Case 3, at Critical Section, Interior Span Side : The loads that occupy the Overhang - Strength V

Note: For this load combination, wind load is included on the noise barrier/fence.

Load Factor, γ_{LL} =	1.35	For M_{LL}	*AASHTO Table 3.4.1-1*
Load Factor, γ_{WS} =	0.40	For M_{WS}	

Dead Load Moments

Total DL Fact. $M_{U(D)} = \eta_i(\gamma_{pDC}(M_{DC1} + M_{DC2} + M_{DC3} + M_{DC4} + M_{DC5} + M_{DC5i}) + \gamma_{pDW} M_{DW})$ =	2.70 k-ft/ft	Same as Part B) above

Live Load Moment

Factored LL + I Moment, $M_{U(LL+I)} = \eta_i \gamma_{LL} M_{(LL+I)}$ =	6.43 k-ft/ft	Same as Part B) above, except with new factor

Noise Barrier/Fence Wind Moment

$M_{WS} = A * P_W (N_H) ((N_H)/2 + B_H + T_L + T_S/2)$ =	0.00 k-ft/ft	
Factored Wind Moment, $M_{U(WS)} = \eta_i \gamma_{WS} M_{WS}$ =	0.00 k-ft/ft	
Total Fact. Moment = $M_{U(D)} + M_{U(LL+I)} + M_{U(WS)}$ =	9.14 k-ft/ft	

Check Moment Capacity of Reinforcement

ϕM_n =	15.97 k-ft/ft	**OK! > Mu**

Check Crack Control

Total Service Moment = $M_D + M_{LL+I} + M_{WS}$ =	6.60 k-ft/ft	
$f_s = M / A_{sTotal} j d$ =	22.98 ksi	
Exposure Factor, γ_e =	0.75 k/in	
Depth From Extreme Tension Fiber to Closest Bar Center, $d_c = C_{Vt} + d_{avg}/2$ =	2.81 in	
Strain Ratio, $\beta_s = 1 + d_c / [0.7 * (h - d_c)]$ =	1.706	Where h is equal to the interior slab thickness T_S
Spacing Limit, $s_{max} = (700 * \gamma_e / \beta_s f_s) - 2 * d_c$ =	7.76 in	
Actual Spacing =	5.50 in	**OK! < Max spa**

D) Results

Location of Critical Section with Additional No Longer Required =		10.0 in	
Additional Cut-off Length = Max of:	Effective Depth, d_e =	5.7 in	AASHTO LRFD 5.11.1.2
	15 d_b =	9.4 in	
	1/20 clear span =	5.3 in	
Total Length of Additional Reinforcing Beyond CL Beam =		20 in	**OK > 25 bar diameters**

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Input Parameters Case 1:

Deck & Beam Dimension Input:

Beam Type (Select From Drop Down List)	Concrete Bulb-Tee	Web Thickness, t_w =	8.0000 in	
Beam Spacing, B =	8.833 ft	Top Flange Width, b_f =	48.0 in	
Slab Overhang, O =	3.500 ft	CL Beam to Span (Interior), I =	1.250 ft	
CL Beam to Curb Line, C =	2.000 ft	CL Beam to Span (Exterior), E =	1.250 ft	
Thickness of Concrete Slab Interior Span, T_S =	8.50 in	Slab Thickness @ Critical Sect. in Overhang, T_O =	8.50 in	
Minimum Thickness of Concrete Slab Overhang, $T_{O\text{-}Min}$ =	8.50 in	Thickness of LMC Overlay, T_L =	0.00 in	

Loading Input:

Uniform Weight of Permanent Form, W_F =	0.000 ksf	Noise Wall/Fence Height, N_H =	0.000 ft	
Uniform Weight of Future Wearing Surface, W_{DW} =	0.060 ksf	Noise Wall/Fence CG to Curb, N_{CG} =	0.000 ft	
Increase/Decrease (LL+I) Depending on Loading =	1.00	Noise Wall Weight, N_W =	0.000 klf	
LL Wheel Load, P =	16.0 kips	Wind Pressure, P_W =	0.000 ksf	
Interior Design Positive Moment, M^+_{LL+I} =	6.29 k-ft/ft	Design Span, S = B - 2 x I =	6.33 ft	
Interior Design Negative Moment, M^-_{LL+I} =	3.71 k-ft/ft	Effective Span, S_{eff} = B - t_w =	8.17 ft	Per AASHTO LRFD 9.7.2.3

Concrete & Reinforcement Input:

Assumed Wearing Surface of Slab, T_W =	1.0 in	Concrete Compressive Strength, f'_c =	4.5 ksi	
Positive Moment, Bottom Reinf. Concrete Cover, C_{Vb} =	1.50 in	Yield Stress of Reinforcing, f_y =	60.0 ksi	
Negative Moment, Top Reinf. Concrete Cover, C_{Vt} =	2.50 in	Modular Ratio, n =	7.13	
Unit Weight of Deck Concrete, γ_{conc} =	0.150 kcf	Critical Section for Add'l Reinf., Interior Slab =	10.0 in	Dist. CL Ext. Beam into Interior
Include Sidewalk Dead Load for Interior Design?	No	Length of Sidewalk from FF of Barrier, L_{SW} =	0.00 ft	
Thickness of Sidewalk in Interior Span, $T_{SW\text{-}I}$ =	0.00 in	Thickness of Sidewalk in Overhang, T_{SW} =	0.00 in	Average Thickness

Positive Moment Bottom Reinforcement	#5 bar spa. @	5.50 in	Abar =0.31in^2	dia. = 0.625 in	
Negative Moment Top Reinforcement	#5 bar spa. @	5.50 in	Abar =0.31in^2	dia. = 0.625 in	
Additional Reinforcement in Overhang	#5 bar spa. @	22.00 in	Abar =0.31in^2	dia. = 0.625 in	

Output:

Interior Design:

Moment Capacity — Adjust Input Values to Meet Design Adequacy for these Critical Moments and Stresses! — Over-reinforced ?

Positive Moment :	ϕM_n =	15.97 k-ft/ft	**OK! > Mu**	M_u =	11.90 k-ft/ft	**OK! < 0.42**
Negative Moment :	ϕM_n =	15.97 k-ft/ft	**OK! > Mu**	M_u =	7.39 k-ft/ft	**OK! < 0.42**

Crack Control

Positive Moment :	spa =	5.50 in	**OK! < Max spa**	Max. spa =	11.27 in	
Negative Moment :	spa =	5.50 in	**OK! < Max spa**	Max. spa =	14.57 in	

Overhang Design:

Part 1) Design Case 1, at Base of Barrier : Horizontal Vehicular Collision Load - Extreme Event Limit State

Moment Capacity	ϕM_n =	21.36 k-ft/ft	**OK! > Mu**	M_u =	17.84 k-ft/ft	**OK! < 0.42**

Part 2) Design Case 1, at Critical Section, Overhang Side : Horizontal Vehicular Collision Load - Extreme Event Limit State

Moment Capacity	ϕM_n =	21.46 k-ft/ft	**OK! > Mu**	M_u =	18.20 k-ft/ft	**OK! < 0.42**

Part 3) Design Case 1, at Critical Section, Interior Span Side : Horizontal Vehicular Collision Load - Extreme Event Limit State

Moment Capacity	fM_n =	21.47 k-ft/ft	**OK! > Mu**	M_u =	13.93 k-ft/ft	**OK! < 0.42**

Part 4) Design Case 3, at Critical Section, Overhang Side : The loads that occupy the Overhang - Strength Limit State I

Moment Capacity	ϕM_n =	19.54 k-ft/ft	**OK! > Mu**	M_u =	1.93 k-ft/ft	
Crack Control	spa =	5.50 in	**OK! < Max spa**	Max. spa =	65.21 in	

Part 5) Design Case 3, at Critical Section, Overhang Side : The loads that occupy the Overhang - Strength Limit State V

Moment Capacity	ϕM_n =	19.54 k-ft/ft	**OK! > Mu**	M_u =	1.93 k-ft/ft	
Crack Control	spa =	5.50 in	**OK! < Max spa**	Max. spa =	65.21 in	

Part 6) Design Case 3, at Critical Section, Interior Span Side : The loads that occupy the Overhang - Strength Limit State I

Moment Capacity	fM_n =	19.54 k-ft/ft	**OK! > Mu**	M_u =	-0.74 k-ft/ft	
Crack Control	spa =	5.50 in	**OK! < Max spa**	Max. spa =	931.25 in	

Part 7) Design Case 3, at Critical Section, Interior Span Side : The loads that occupy the Overhang - Strength Limit State V

Moment Capacity	fM_n =	19.54 k-ft/ft	**OK! > Mu**	M_u =	-0.05 k-ft/ft	
Crack Control	spa =	5.50 in	**OK! < Max spa**	Max. spa =	931.25 in	

Part 8) Compute Additional Reinforcing Overhang Cutoff Length Requirement

Total Length of Add'l Reinforcement past CL Beam =			**20.0 in**			**OK > 25 bar diameters**
Moment Capacity	fM_n =	17.58 k-ft/ft	**OK! > Mu**	M_u =	15.58 k-ft/ft	**OK! < 0.42**
Moment Capacity	fM_n =	15.97 k-ft/ft	**OK! > Mu**	M_u =	0.76 k-ft/ft	**OK! < Max spa**
Moment Capacity	fM_n =	15.97 k-ft/ft	**OK! > Mu**	M_u =	1.23 k-ft/ft	**OK! < Max spa**

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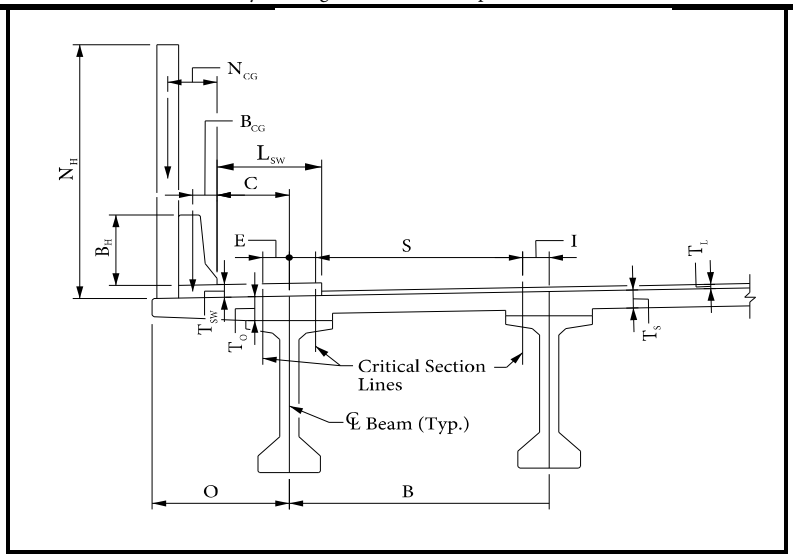

INTERIOR DESIGN OF CONCRETE SLAB:

1) Dead Load Moments

W_{DC} =Total Uniform Load of Slab, Perm. Form and Overlay

$W_{DC} = (T_S + T_L) / 12 * \gamma_{conc} + W_F =$ 0.11 ksf

W_{sw} = Uniform Load of Sidewalk

$W_{sw} = (T_{sw}) / 12 * \gamma_{conc} =$ 0.00 ksf

Note: Overlay ignored in W_{DC} if sidewalk within interior bay.

Load Factor, γ_{pDC} =	1.25	For W_{DC}	*AASHTO Table 3.4.1-1 Table 3.4.1-2*
Load Factor, γ_{pDW} =	1.50	For W_{DW}	
Load Factor, γ_{LL} =	1.75	For M_{LL+I}	
Resistance Factor, ϕ =	0.90	For Flexure	*AASHTO 5.5.4.2.1*
Load Modifier, η_i =	1	For All Loads	*AASHTO 1.3.5*

Positive and Negative Moment

$M_D = (W_{DC} + W_{DW} + W_{sw}) S^2 / 10 =$ 0.67 k-ft/ft

Equations are derived from $w l^2 / 8$ x 0.80 continuity factor

$M_{U(D)} = (\eta_i \gamma_{pDC} (W_{DC} + W_{sw}) + \eta_i \gamma_{pDW} W_{DW}) S^2 / 10 =$ 0.89 k-ft/ft

2) Live Load Moments

Positive Moment $M_{U(LL+I)} = \eta_i \gamma_{LL} M^+_{LL+I} =$ 11.01 k-ft/ft

Negative Moment $= \eta_i \gamma_{LL} M^-_{LL+I} =$ 6.49 k-ft/ft

3) Design of Reinforcement

Unit Width of Concrete Slab, b = 12.00 in

Positive Moment

$A_{s\ PRV'D}$ =	0.676 in^2/ft	
Effective Depth, $d = T_S - T_W - C_{Vb} - d_{bar}/2$ =	5.688 in	
$M_{U(D+LL+I)} = M_{U(D)} + M_{U(LL+I)}$ =	11.90 k-ft/ft	
$\rho_{PRV'D}$ =	0.00991	
$a = A_{s(Bott)} f_y / 0.85 f'_c b$ =	0.884 in	
$\phi M_n = \phi A_s f_y (d - a/2)$ =	15.97 k-ft/ft	**OK! > Mu**
$R_N = M_U / (\phi b d^2)$ =	0.41 ksi	
$\rho_{REQ'D} = .85 (f'_c / f_y) \{1 - [1 - 2 R_N / (.85 f'_c)]^{.5}\}$ =	0.00722	
$A_{s\ REQ'D} = \rho_{REQ'D} b d$ =	0.493 in^2/ft	
Req'd Bar Spacing $= (A_{bar(Bott)} / A_{s\ REQ'D})$ x 12 =	7.547 in	
Check that the section is not over-reinforced, AASHTO 5.7.3.3.1: $c / d = a / \beta_1 / d$ =	0.188	**OK! < 0.42**

Negative Moment

$A_{s\ PRV'D}$ =	0.676 in^2/ft	
Effective Depth, $d = T_S - C_{Vt} - d_{bar}/2$ =	5.688 in	
$M_{U(D+LL+I)} = M_{U(D)} + M_{U(LL+I)}$ =	7.39 k-ft/ft	
$\rho_{PRV'D}$ =	0.00991	
$a = A_{s(Top)} f_y / 0.85 f'_c b$ =	0.884 in	
$\phi M_n = \phi A_s f_y (d - a/2)$ =	15.97 k-ft/ft	**OK! > Mu**
$R_N = M_U / (\phi b d^2)$ =	0.25 ksi	
$\rho_{REQ'D} = .85 (f'_c / f_y) \{1 - [1 - 2 R_N / (.85 f'_c)]^{.5}\}$ =	0.00438	
$A_{s\ REQ'D} = \rho_{REQ'D} b d$ =	0.299 in^2/ft	
Req'd Bar Spacing $= (A_{bar(Top)} / A_{s\ REQ'D})$ x 12 =	12.447 in	
Check that the section is not over-reinforced, AASHTO 5.7.3.3.1: $c / d = a / \beta_1 / d$ =	0.188	**OK! < 0.42**

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Longitudinal Distribution Steel

Bottom

AASHTO 9.7.3.2 - For primary reinforcement perpendicular to traffic

% Distribution Steel = 220 / $(S_{eff})^{.5}$ < 67% = 76.98% **Use 67.00%**

Area of Steel, $A_{s(Dist)}$ = % x $A_{s,eff.(Bott)}$ = 0.453 in^2

Required Spacing = (A_{bar} / A_{sDist}) x 12 for #5 bars = 8.21 in

Top ~~**Use #5's @ 8"**~~ **Use #5 @ 13" per ODOT BDM Fig. 302.2.2-1**

AASHTO 5.10.8 & 5.10.3.2

Area of Steel, A_s >=1.30bh / 2(b+h)f_y = 0.054 in^2/ft

0.11 <= A_s <= 0.60 0.110 in^2

Required Spacing = (A_{bar} / A_{sDist}) x 12 for #4 bars = 21.82 in

Maximum Spacing = 3.0 x T_S < 18 in. = 18.00 in

~~**Use #4's @ 8" Min. for Crack Control**~~ **Use #4 @ 12.5" per ODOT BDM Fig. 302.2.2-1**

4) Minimum Reinforcement

(AASHTO 5.4.2.6,& 5.7.3.3.2)

For Normal Weight Concrete $f_r = 0.37(f'_c)^{.5}$ = 0.78 ksi

$I_g = (1/12)*bh^3$ = 421.9 in^4

$y_t = h/2$ = 3.75 in

1.2 $M_{cr} = f_r (I_g / y_t)$ = 8.83 k-ft/ft

Positive Moment

ϕM_n must be greater than 1.2M_{cr}: **OK!**

If 1.2 M_{cr} > M_u, then check that ϕM_n need not be greater than 4/3 M_u

4/3$M_{U(D+LL+I)}$ = 15.87 k-ft/ft **Check Not Needed !!**

Negative Moment

ϕM_n must be greater than 1.2M_{cr}: **OK!**

If 1.2 M_{cr} > M_u, then check that ϕM_n need not be greater than 4/3 M_u

4/3$M_{U(D+LL+I)}$ = 9.85 k-ft/ft **Check Not Needed !!**

5) Check Crack Control

(AASHTO 5.7.3.4)

Exposure Factor, γ_e = 0.75 k/in

Compute Service Load Stress:

Note: Based on previous calculations, the section does not exceed the cracked moment per AASHTO provisions and therefore is assumed uncracked.

Positive Moment

$M_{D+LL+I} = M_D + M_{LL+I}$ = 6.96 k-ft/ft

k =$[(\rho n)^2+2 \rho n]^{.5} - \rho n$ = 0.312 j = 1 - k/3 = 0.896

C = T = M_{D+LL+I} / (j d) = 16.38 k

$f_s = T / A_s$ = 24.22 ksi

f_c = 2 C / (b k d) = 1.539 ksi

Negative Moment

$M_{D+LL+I} = M_D + M_{LL+I}$ = 4.38 k-ft/ft

k =$[(\rho n)^2 + 2 \rho n]^{.5} - \rho n$ = 0.312 j = 1 - k/3 = 0.896

C = T = M_{D+LL+I} / (j d) = 10.31 k

$f_s = T / A_s$ = 15.24 ksi

f_c = 2 C / (b k d) = 0.968 ksi

Determine Maximum Allowable Spacing:

Positive Moment

Depth From Extreme Tension Fiber to Closest Bar Center, $d_c = C_{Vb} + d_{bar}/2$ = 1.813 in

Strain Ratio, $\beta_s = 1 + d_c / [0.7 * (h - d_c)]$ = 1.455 Where h is equal to the slab thick. minus wearing surface, $T_s - T_w$

Spacing limit, $s_{max} = (700 * \gamma_e / \beta_s f_s) - 2 * d_c$ = 11.27 in

Actual Spacing = 5.50 in **OK! < Max spa**

Negative Moment

Depth From Extreme Tension Fiber to Closest Bar Center, $d_c = C_{Vt} + d_{bar} / 2$ = 2.81 in

Strain Ratio, $\beta_s = 1 + d_c / [0.7 * (h - d_c)]$ = 1.706 Where h is equal to the slab thickness, T_s

Spacing limit, $s_{max} = (700 * \gamma_e / \beta_s f_s) - 2 * d_c$ = 14.57 in

Actual Spacing = 5.50 in **OK! < Max spa**

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Backchecked by	AKS	Date	2/3/2011	**1/16/2012**

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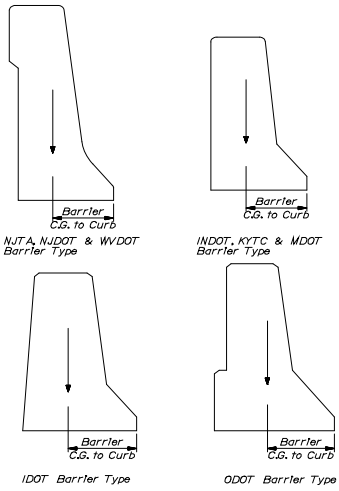
OVERHANG DESIGN: AASHTO A13.4

Concrete Barrier Properties

DOT or Project Specific Barrier Type	ODOT - 57in.
Barrier Area =	4.940 ft^2
Barrier CG to Curb Line, B_{CG} =	0.946 ft
Barrier Height, B_H =	4.750 ft
Barrier Weight, B_W =	0.741 klf
Equivalent Width of Barrier as a Uniform Section	
Modified "w" Value To Be Used? =	ODOT - 57in.
w = Barrier Area / Barrier Height =	1.040 ft



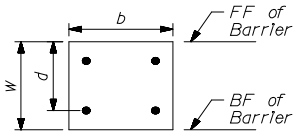
Step 1 -Determine Moment Capacity of Barrier

AASHTO A13.3.1

A) Capacity of Barrier about an Axis Parallel to the Longitudinal Axis of the Bridge:

Consider 1' Section of Barrier

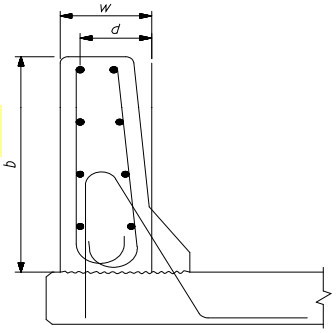
f'_c =	4.5 ksi	
Yield Stress of Reinforcement, f_y =	60.0 ksi	
Vertical Reinforcement	#5 bar spa. @	11.00 in
A_s =	0.338 in^2/ft	
Dia. =	0.625 in	
Cover, C_V =	2.0 in	
b =	12.00 in	
d = w - Cv - Dia. / 2 =	10.168	
$M_c = \phi M_n = \phi\ A_s f_y (d - a/2) / b$ =	16.8 k-ft/ft	



B) Capacity of Barrier about its Vertical Axis:

Consider Equivalent Width Barrier

Top Half Reinforcement	1	#6 bars
Bottom Half Reinforcement	6	#5 bars
A_s =	2.300 in^2	
Diameter of Vertical Reinforcement =	0.625 in	
Average Diameter of Longitudinal Reinforcement =	0.688 in	
Cover, C_V =	2.0 in	
(In this instance is equal to height of barrier) b =	57.00 in	
d = w - Cv - Dia_V - Dia_L / 2 =	9.511	
$M_w = \phi M_n = \phi\ A_s f_y (d - a/2)$ =	105.7 k-ft	



Step 2 - Determine Barrier Resistance to Transverse Loads, R_w:

A) For Impacts Within a Wall Segment: AASHTO A13.3.1

$R_w = (2 / (2L_c - L_t))(8\ M_b + 8\ M_w + M_c\ L_c^2 / H)$

Where:	M_b =	0.0 k-ft	*(additional flexural resistance at top of barrier, if any)*
	Height of Barrier, H =	4.750 ft	
	Test Level =	TL-2	
	L_t =	4.0 ft	Table A13.2-1 for appropriate Test Level
	Distance Between Vertical Open Joints in Barrier =	25.000 ft	
	$L_c = L_t / 2 + [(L_t / 2)^2 + 8\ H\ (M_b + M_w) / M_c]^{0.5}$ =	17.6 ft	**Use 17.6 ft**
	R_w =	124.53 kips	Max. 1.25 x F_t from Table A13.2-1
	USE R_w =	33.75 kips	**Does Not Govern!**

AASHTO A13.4.2

For Design Case 1, the deck overhang may be designed to provide a flexural resistance, M_s (k-ft/ft) which, acting coincident with the tensile force P_T (k/ft) specified herein, exceeds M_c of the parapet **at its base**.

Design Axial Tensile Force, $P_T = R_w / (L_c + 2\ H)$ = 1.25 k/ft

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B) For Impacts at End of Wall or at Joint: AASHTO A13.3.1

$R_w = (2 / (2L_c - L_t))(M_b + M_w + M_c L_c^2 / H)$

Where: M_b =	0.0 k-ft	*(additional flexural resistance at top of barrier, if any)*
Height of Barrier, H =	4.750 ft	
L_t =	4.0 ft	Table A13.2-1 for appropriate Test Level
Distance Between Vertical Open Joints in Barrier =	25.000 ft	
$L_c = L_t / 2 + [((L_t / 2)^2 + H (M_b + M_w) / M_c)]^{0.5}$ =	7.8 ft	**Use 7.8 ft**
R_w =	55.37 kips	Max. 1.25 x F_t from Table A13.2-1
USE R_w =	33.75 kips	**Does Not Govern!**

AASHTO A13.4.2

For Design Case 1, the deck overhang may be designed to provide a flexural resistance, M_s (k-ft/ft) which, acting coincident with the tensile force P_T (k/ft) specified herein, exceeds M_c of the parapet **at its base**.

Design Axial Tensile Force, $P_T = R_w / (L_c + 2 H)$ = 1.95 k/ft

Step 3 - Evaluate Various Extreme Event and Strength Limit State Cases:

Part 1) Design Case 1, at Base of Barrier : Horizontal Vehicular Collision Load - Extreme Event Limit State

Dead Load Moments

Uniform Load of Slab, $W_{DC1} = (T_{O\text{-}Min} + T_O)/2 / 12 * \gamma_{conc}$ = 0.11 ksf

Load Factor, γ_{pDC} = 1.25 For M_{DC} *AASHTO Table 3.4.1-1*

Uniform Load of Overlay, W_{DC2}, and Overlay Moment, M_{DC2} not present

Load Modifier, η_i = 1.0 For All Loads

Moments

Resistance Factor, ϕ = 1.0 For Extreme Event Limit State

Slab Moment, $M_{DC1} = W_{DC1} (O - C)^2 / 2$ =	0.12 k-ft/ft
Barrier Moment, $M_{DC3} = B_W B_{CG}$ =	0.70 k-ft/ft
Noise Barrier/Fence Moment, $M_{DC4} = N_W N_{CG}$ =	0.00 k-ft/ft
Total Dead Load Service Moment =	0.82 k-ft/ft
Total DL Factored $M_{U(D)} = \eta_i \gamma_{pDC} (M_{DC1} + M_{DC3} + M_{DC4})$ =	1.03 k-ft/ft
Total Design Factored Moment, $M_u = M_c + M_{U(D)}$ =	17.84 k-ft/ft

Note: In an Extreme Event Limit State load combination, wind on structure is not included, and therefore is not included as a load on the noise barrier/fence.

Check Moment Capacity of Reinforcement

For a section under moment and axial tension, P_T, the nominal resistance, M_n, may be calculated as: $\phi M_n = \phi[T(d - a/2) - P_T(d/2 - a/2)]$

$A_{s(Top)}$ =	0.676 in^2/ft	$A_{s(Add'l)}$ =	0.169 in^2/ft
Avg. Dia., $d_{avg} = (A_{s(Top)} Dia_{Top} + A_{s(Add'l)} Dia_{Add'l}) / A_{sTotal}$ =	0.625 in	b =	12.00 in
Effective Depth, $d = (T_{O\text{-}Min} + T_O)/2 - C_{Vt} - d_{avg}/2$ =	5.688 in	$\rho_{PRV'D}$ =	0.01239
$T = (A_{s(Top)} + A_{s(Add'l)}) f_y$ =	50.73 k/ft		
Compression in Concrete, $C = T - P_T$ =	48.78 k/ft	Where P_T is the Design Axial Tensile Force from Step 2 Above	
$a = C / b \beta_1 f'_c$ =	1.095 in		
ϕM_n =	21.36 k-ft/ft	**OK! > Mu**	
Check that the section is not over-reinforced, AASHTO 5.7.3.3.1: $c / d = a / \beta_1 / d$ =	0.233	**OK! < 0.42**	

Part 2) Design Case 1, at Critical Section, Overhang Side : Horizontal Vehicular Collision Load - Extreme Event Limit State

Assume the distribution of collision loads increases by 0.8X as the analysis point moves X ft away from the barrier. 0.8 corresponds to a distribution angle of 22° for the increase of the force distribution from the barrier face similar to the AASHTO Standard LFD Specifications.

Design Moment due to Collision Acting at Critical Section: $M_{CS} = M_c L_c / (L_c + 0.8X)$

Where: M_c =	16.8 k-ft/ft
L_c =	17.6 ft
$X = C - E$ =	0.750 ft
M_{CS} =	16.26 k-ft/ft

Design Axial Tensile Force Acting at Critical Section:

$P_{CS} = R_w / (L_c + 2 H + 0.8X)$ = 1.22 k/ft

Dead Load Moments

Uniform Load of Slab, $W_{DC1} = (T_{O\text{-}Min} + T_O)/2 / 12 * \gamma_{conc}$ = 0.11 ksf

Load Factor, γ_{pDC} = 1.25 For M_{DC} *AASHTO Table 3.4.1-1*

Uniform Load of Overlay, $W_{DC2} = T_L / 12 * \gamma_{conc}$ = 0.00 ksf

Load Factor, γ_{pDW} = 1.50 For M_{DW}

Uniform Load of Sidewalk, $W_{DC5} = T_{SW}/12 * \gamma_{conc}$ = 0.00 ksf

Load Modifier, η_i = 1.0 For All Loads

Moments

Resistance Factor, ϕ = 1.0 For Extreme Event Limit State

Slab Moment, $M_{DC1} = W_{DC1} (O - E)^2 / 2$ =	0.27 k-ft/ft
Overlay Moment, $M_{DC2} = W_{DC2} (C - E)^2 / 2$ =	0.00 k-ft/ft
Barrier Moment, $M_{DC3} = B_W (B_{CG} + C - E)$ =	1.26 k-ft/ft
Noise Barrier/Fence Moment, $M_{DC4} = N_W (N_{CG} + C - E)$ =	0.00 k-ft/ft
Sidewalk Moment, $M_{DC5} = W_{DC5} (C - E)^2 / 2$ =	0.00 k-ft/ft
FWS Moment, $M_{DW} = W_{DW} (C - E)^2 / 2$ =	0.02 k-ft/ft
Total Dead Load Service Moment =	1.54 k-ft/ft
Total DL Fact. $M_{U(D)} = \eta_i(\gamma_{pDC} (M_{DC1} + M_{DC2} + M_{DC3} + M_{DC4} + M_{DC5}) + \gamma_{pDW} M_{DW})$ =	1.93 k-ft/ft
Total Design Factored Moment, $M_u = M_{CS} + M_{U(D)}$ =	18.20 k-ft/ft

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For a section under moment and axial tension, P_{CS}, the nominal resistance, M_n, may be calculated as: $\phi M_n = \phi[T(d - a/2) - P_{CS}(d/2 - a/2)]$

$A_{s(Top)}$ =	0.676 in^2/ft	$A_{s(Add'l)}$ =	0.169 in^2/ft	
Avg. Dia., $d_{avg} = (A_{s(Top)}\ Dia_{Top} + A_{s(Add'l)}\ Dia_{Add'l}) / A_{sTotal}$ =	0.625 in	b =	12.00 in	
Effective Depth, $d = T_O - C_{Vt} - d_{avg}/2$ =	5.688 in	$\rho_{PRV'D}$ =	0.01239	
$T = (A_{s(Top)} + A_{s(Add'l)})\ f_y$ =	50.73 k/ft			
Compression in Concrete, $C = T - P_{CS}$ =	49.51 k/ft	Where P_{CS} is the Design Axial Tensile Force from above		
$a = C / b\ \beta_1\ f'_c$ =	1.111 in			
ϕM_n =	21.46 k-ft/ft	**OK! > Mu**		
Check that the section is not over-reinforced, AASHTO 5.7.3.3.1: $c / d = a / \beta_1 / d$ =	0.237	**OK! < 0.42**		

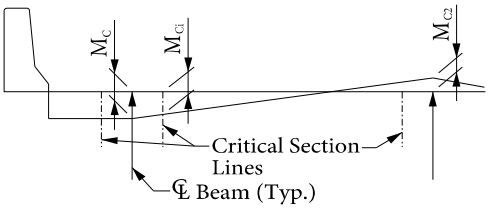
Part 3) Design Case 1, at Critical Section, Interior Span Side : Horizontal Vehicular Collision Load - Extreme Event Limit State

Assume the collision moment at the first interior beam, $M_{c2} = (O / B) * M_c$

Interpolate to find the collision moment at the interior side, $M_{ci} = M_c - I * (M_c + M_{c2}) / B$

$A = 1 - (\text{Min. of bf / 3 or 1.25 ft}) * (1 + O / B) / B$ = 0.802

Design Moment due to Collision Acting at Critical Section: $M_{CS} = M_{ci}\ L_c / (L_c + 0.8X)$



M_{c2} =	6.7 k-ft/ft
M_{ci} =	13.5 k-ft/ft
L_c =	17.6 ft
$X = C + E$ =	3.250 ft
M_{CS} =	11.76 k-ft/ft

Design Axial Tensile Force Acting at Critical Section:

$P_{CS} = R_w / (L_c + 2\ H + 0.8X)$ = 1.14 k/ft

Dead Load Moments

Uniform Load of OH Slab, $W_{DC1} = (T_{O\text{-}Min} + T_O)/2/\ 12 * \gamma_{conc}$ =	0.11 ksf	Load Factor, γ_{pDC} =	1.25	For M_{DC}	*AASHTO Table 3.4.1-1*
Uniform Load of Overlay, $W_{DC2} = T_L / 12 * \gamma_{conc}$ =	0.00 ksf	Load Factor, γ_{pDW} =	1.5	For M_{DW}	
Uniform Load of Interior Slab, $W_{DC1i} = T_{Si} / 12 * \gamma_{conc}$ =	0.11 ksf	Load Modifier, η_i =	1.0	For All Loads	
Uniform Load of Sidewalk, $W_{DC5} = T_{SW} / 12 * \gamma_{conc}$ =	0.00 ksf	Resistance Factor, ϕ =	1.0	For Extreme Event Limit State	
Uniform Load of Interior Sidewalk, $W_{DC5i} = T_{SWi} / 12 * \gamma_{conc}$ =	0.00 ksf				

Moments

Exterior Slab Moment, $M_{DC1} = A\ W_{DC1}\ (O)^2 / 2$ =	0.52 k-ft/ft
Exterior Overlay Moment, $M_{DC2} = A\ W_{DC2}\ (C)^2 / 2$ =	0.00 k-ft/ft
Interior Slab Moment, $M_{DC1i} = 0.4W_{DC1i}\ (B)(E) - W_{DC1i}\ (E)^2 / 2$ =	-0.39 k-ft/ft
Interior Overlay Moment, $M_{DC2i} = 0.4W_{DC2}\ (B)(E) - W_{DC2}\ (E)^2 / 2$ =	0.00 k-ft/ft
Barrier Moment, $M_{DC3} = A\ B_W\ (B_{CG} + C)$ =	1.75 k-ft/ft
Noise Barrier/Fence Moment, $M_{DC4} = A\ N_W\ (N_{CG} + C)$ =	0.00 k-ft/ft
Sidewalk Moment, $M_{DC5} = A\ W_{DC5}\ (C)^2 / 2$ =	0.00 k-ft/ft
Interior Sidewalk Moment, $M_{DC5i} = 0.5\ W_{DC5i}\ (L_{SW} - C)(I) - W_{DC5i}\ (I)^2 / 2$ =	0.00 k-ft/ft
FWS Moment, $M_{DW} = A\ W_{DW}\ (C)^2 / 2$ =	0.10 k-ft/ft
Interior FWS Moment, $M_{DWi} = 0.4M_{DW}\ (B)(E) - M_{DW}\ (E)^2 / 2$ =	-0.22 k-ft/ft
Total Dead Load Service Moment =	1.77 k-ft/ft
Total Fact. $M_{U(D)} = \eta_i(\gamma_{pDC}(M_{DC1}+M_{DC2}+M_{DC1i}+M_{DC2i}+M_{DC3}+M_{DC4}+M_{DC5}+M_{DC5i})+\gamma_{pDW}(M_{DW}+M_{DWi})$ =	2.18 k-ft/ft
Total Design Factored Moment, $M_u = M_{CS} + M_{U(D)}$ =	13.93 k-ft/ft

For a section under moment and axial tension, P_{CS}, the nominal resistance, M_n, may be calculated as: $\phi M_n = \phi[T(d - a/2) - P_{CS}(d/2 - a/2)]$

$A_{s(Top)}$ =	0.676 in^2/ft	$A_{s(Add'l)}$ =	0.169 in^2/ft	
Avg. Dia., $d_{avg} = (A_{s(Top)}\ Dia_{Top} + A_{s(Add'l)}\ Dia_{Add'l}) / A_{sTotal}$ =	0.625 in	b =	12.00 in	
Effective Depth, $d = T_{Si} - C_{Vt} - d_{avg}/2$ =	5.688 in	$\rho_{PRV'D}$ =	0.01239	
$T = (A_{s(Top)} + A_{s(Add'l)})\ f_y$ =	50.73 k/ft			
Compression in Concrete, $C = T - P_{CS}$ =	49.59 k/ft	Where P_{CS} is the Design Axial Tensile Force from above		
$a = C / b\ \beta_1\ f'_c$ =	1.113 in			
ϕM_n =	21.47 k-ft/ft	**OK! > Mu**		
Check that the section is not over-reinforced, AASHTO 5.7.3.3.1: $c / d = a / \beta_1 / d$ =	0.237	**OK! < 0.42**		

Part X) Design Case 2: Vertical Vehicular Collision Load - Extreme Event Limit State

This case does not control design for concrete barriers and was therefore not checked.

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Part 4) Design Case 3, at Critical Section, Overhang Side : The loads that occupy the Overhang - Strength Limit State I

Note: For the noise barrier/fence, wind load is not included under this load combination.

Load Factor, γ_{pDC} =	1.25	For M_{DC}	*AASHTO Table 3.4.1-1*
Load Factor, γ_{pDW} =	1.5	For M_{DW}	
Load Factor, γ_{LL} =	1.75	For M_{LL}	
Load Modifier, η_i =	1.00	For All Loads	
Resistance Factor, ϕ =	0.9	For Strength Limit State	

Dead Load Moments

Uniform Load of Slab, $W_{DC1} = (T_{O\text{-}Min} + T_O)/2 / 12 * \gamma_{conc}$ =	0.11 ksf
Uniform Load of Overlay, $W_{DC2} = T_L / 12 * \gamma_{conc}$ =	0.00 ksf
Uniform Load of Sidewalk, $W_{DC5} = T_{sw} / 12 * \gamma_{conc}$ =	0.00 ksf
Moments	
Slab Moment, $M_{DC1} = W_{DC1} (O - E)^2 / 2$ =	0.27 k-ft/ft
Overlay Moment, $M_{DC2} = W_{DC2} (C - E)^2 / 2$ =	0.00 k-ft/ft
Barrier Moment, $M_{DC3} = B_W (B_{CG} + C - E)$ =	1.26 k-ft/ft
Noise Barrier/Fence Moment, $M_{DC4} = N_W (N_{CG} + C - E)$ =	0.00 k-ft/ft
Sidewalk Moment, $M_{DC5} = W_{DC5} (C - E)^2 / 2$ =	0.00 k-ft/ft
FWS Moment, $M_{DW} = W_{DW} (C - E)^2 / 2$ =	0.02 k-ft/ft
Total Dead Load Service Moment =	1.54 k-ft/ft
Total DL Fact. $M_{U(D)} = \eta_i(\gamma_{pDC}(M_{DC1} + M_{DC2} + M_{DC3} + M_{DC4} + M_{DC5}) + \gamma_{pDW} M_{DW})$ =	1.93 k-ft/ft

Live Load Moment

Multiple Presence Factor, m =	0.65	AASHTO Table 3.6.1.1.2-1 for 2 Lanes Loaded
Dynamic Load Allowance (Impact), IM =	1.33	AASHTO 3.6.2.1
X = C - E - 1ft =	0.000 ft	
Equivalent Strip Width for Live Load, W = 45 + 10 X =	45.000 in	AASHTO Table 4.6.2.1.3-1 (X is in feet)
Wheel Load, P =	16.0 kips	
M_{LL+I} = m (IM) P X / W =	0.00 k-ft/ft	
Factored LL + I Moment, $M_{U(LL+I)} = \eta_i \gamma_{LL} M_{(LL+I)}$ =	0.00 k-ft/ft	

Check Moment Capacity of Reinforcement

$A_{s(Top)}$ =	0.676 in^2/ft	$A_{s(Add'l)}$ =	0.169 in^2/ft
Avg. Dia., $d_{avg} = (A_{s(Top)} Dia_{Top} + A_{s(Add'l)} Dia_{Add'l}) / A_{sTotal}$ =	0.625 in	b =	12.00 in
Effective Depth, $d = T_O - C_{Vt} - d_{avg}/2$ =	5.688 in	$\rho_{PRV'D}$ =	0.01239
$M_{U(D+LL+I)} = M_{U(D)} + M_{U(LL+I)}$ =	1.93 k-ft/ft		
$a = A_{sTotal} f_y / 0.85 f'_c b$ =	1.105 in		
$\phi M_n = \phi A_{sTotal} f_y (d - a/2)$ =	19.54 k-ft/ft	**OK! > Mu**	

Check Crack Control

(AASHTO 5.7.3.4)

Total Service $M_{D+LL+I} = M_D + M_{LL+I}$ =	1.54 k-ft/ft		
$k = [(\rho n)^2 + 2 \rho n]^{.5} - \rho n$ =	0.341	j = 1 - 1/3*k =	0.886
$C = T = M_{D+LL+I} / (j d)$ =	3.7 k		
$f_s = T / A_{sTotal}$ =	4.34 ksi		
$f_c = 2 C / (b k d)$ =	0.32 ksi		
Exposure Factor, γ_e =	0.75 k/in		
Depth From Extreme Tension Fiber to Closest Bar Center, $d_c = C_{Vt} + d_{avg}/2$ =	2.81 in		
Strain Ratio, $\beta_s = 1 + d_c / [0.7 * (h - d_c)]$ =	1.706	Where h is equal to the maximum overhang slab thickness T_O	
Spacing Limit, $s_{max} = (700 * \gamma_e / \beta_s f_s) - 2 * d_c$ =	65.21 in		
Actual Spacing =	5.50 in	**OK! < Max spa**	

Part 5) Design Case 3, at Critical Section, Overhang Side : The loads that occupy the Overhang - Strength Limit State V

Note: For this load combination, wind load is included on the noise barrier/fence.

Load Factor, γ_{LL} =	1.35	For M_{LL}	*AASHTO Table 3.4.1-1*
Load Factor, γ_{WS} =	0.40	For M_{WS}	

Dead Load Moments

Total DL Fact. $M_{U(D)} = \eta_i(\gamma_{pDC}(M_{DC1} + M_{DC2} + M_{DC3} + M_{DC4} + M_{DC5}) + \gamma_{pDW} M_{DW})$ =	1.93 k-ft/ft	Same as Part 4)

Live Load Moment

Factored LL + I Moment, $M_{U(LL+I)} = \eta_i \gamma_{LL} M_{(LL+I)}$ =	0.00 k-ft/ft	Same as Part 4) above, except with new factor

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LRFD Design - One Course Construction - Summary of Design Criteria and Output — RIGHT OVERHANG

Noise Barrier/Fence Wind Moment

$M_{WS} = P_W (N_H - B_H) ((N_H - B_H)/2 + B_H + T_L + T_O/2) =$	0.00 k-ft/ft	
Factored Wind Moment, $M_{U(WS)} = \eta_i \gamma_{WS} M_{WS} =$	0.00 k-ft/ft	
Total Fact. Moment = $M_{U(D)} + M_{U(LL+I)} + M_{U(WS)} =$	1.93 k-ft/ft	

Check Moment Capacity of Reinforcement

$\phi M_n =$	19.54 k-ft/ft	**OK! > Mu**

Check Crack Control

Total Service Moment = $M_D + M_{LL+I} + M_{WS} =$	1.54 k-ft/ft	
$f_s = M / A_{sTotal}\, j\, d =$	4.34 ksi	
Exposure Factor, $\gamma_e =$	0.75 k/in	
Depth From Extreme Tension Fiber to Closest Bar Center, $d_c = C_{Vt} + d_{avg}/2 =$	2.81 in	
Strain Ratio, $\beta_s = 1 + d_c / [0.7 * (h-d_c)]$	1.706	Where h is equal to the max overhang slab thickness T_O
Spacing Limit, $s_{max} = (700 * \gamma_e / \beta_s f_s) - 2 * d_c =$	65.21 in	
Actual Spacing =	5.50 in	**OK! < Max spa**

Part 6) Design Case 3, at Critical Section, Interior Span Side : The loads that occupy the Overhang - Strength Limit State I

Note: For the noise barrier/fence, wind load is not included under this load combination.

Load Factor, $\gamma_{pDC} =$	1.25	For M_{DC}	*AASHTO Table 3.4.1-1*
Load Factor, $\gamma_{pDW} =$	1.5	For M_{DW}	
Load Factor, $\gamma_{LL} =$	1.75	For M_{LL}	
Load Modifier, $\eta_i =$	1.05	For All Loads	
Resistance Factor, $\phi =$	0.9	For Strength Limit State	

Dead Load Moments

Uniform Load of OH Slab, $W_{DC1} = (T_{O\text{-}Min} + T_O) / 2 / 12 * \gamma_{conc} =$	0.11 ksf	
Uniform Load of Overlay, $W_{DC2} = T_L / 12 * \gamma_{conc} =$	0.00 ksf	
Uniform Load of Interior Slab, $W_{DC1\text{-}i} = T_S / 12 * \gamma_{conc} =$	0.11 ksf	
Uniform Load of Sidewalk, $W_{DC5} = T_{SW} / 12 * \gamma_{conc} =$	0.00 ksf	
Uniform Load of Interior Sidewalk, $W_{DC5i} = T_{SWi} / 12 * \gamma_{conc} =$	0.00 ksf	
Moments		
Slab Moment, $M_{DC1} = A\, W_{DC1}\, (O)^2 / 2 =$	0.52 k-ft/ft	
Overlay Moment, $M_{DC2} = A\, W_{DC2}\, (C)^2 / 2 =$	0.00 k-ft/ft	
Interior Slab Moment, $M_{DC1i} = 0.4 W_{DC1i}\, (B)(E) - W_{DC1i}\, (E)^2 / 2 =$	-0.39 k-ft/ft	
Interior Overlay Moment, $M_{DC2\text{-}i} = 0.4 W_{DC2}\, (B)(E) - W_{DC2}\, (E)_2 / 2 =$	0.00 k-ft/ft	
Barrier Moment, $M_{DC3} = A\, B_W\, (B_{CG} + C) =$	1.75 k-ft/ft	
Noise Barrier/Fence Moment, $M_{DC4} = A\, N_W\, (N_{CG} + C) =$	0.00 k-ft/ft	
Sidewalk Moment, $M_{DC5} = A\, W_{DC5}\, (C)^2 / 2 =$	0.00 k-ft/ft	
Interior Sidewalk Moment, $M_{DC5i} = 0.5\, W_{DC5i}\, (L_{SW} - C)(I) - W_{DC5i}\, (I)^2 / 2 =$	0.00 k-ft/ft	
FWS Moment, $M_{DW} = A\, W_{DW}\, (C)^2 / 2 =$	0.10 k-ft/ft	
FWS Moment, $M_{DWi} = 0.4 W_{DW}\, (B)(E) - W_{DW}\, (E)^2 / 2 =$	-0.22 k-ft/ft	
Total Dead Load Service Moment =	1.77 k-ft/ft	
Total Fact. $M_{U(D)} = \eta_i(\gamma_{pDC}(M_{DC1}+M_{DC2}+M_{DC1i}+M_{DC2i}+M_{DC3}+M_{DC4}+M_{DC5}+M_{DC5i})+\gamma_{pDW}(M_{DW}+M_{DWi}) =$	2.29 k-ft/ft	

Live Load Moment

Multiple Presence Factor, m =	1.20	AASHTO Table 3.6.1.1.2-1 for 1 Lane Loaded
Dynamic Load Allowance (Impact), IM =	1.33	AASHTO 3.6.2.1
If C > 2' then M_{LL} =	[P x ((C - 2) + E) - (P x (B + (C - 2)) + P x (B - (6 - (C - 2)))) / B x E] + P x (E - (2 - C))	
Else M_{LL} =	[-P x ((B - (2 - C)) + (B - (2 - C) - 6)) / B x E] + P x (E - (2 - C))	
M_{LL} =	-6.42 k-ft	
Equivalent Strip Width for Live Load, W = 45 + 10 X =	74.500 in	AASHTO Table 4.6.2.1.3-1 (S is in feet)
Wheel Load, P =	16.0 kips	
$M_{LL+I} = m\, (IM)\, M_{LL} / W =$	-1.65 k-ft/ft	
Factored LL + I Moment, $M_{U(LL+I)} = \eta_i \gamma_{LL} M_{(LL+I)} =$	-3.03 k-ft/ft	

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Check Moment Capacity of Reinforcement

$A_{s(Top)}$ =	0.676 in^2/ft	$A_{s(Add'l)}$ =	0.169 in^2/ft	
Avg. Dia., $d_{avg} = (A_{s(Top)}\ Dia_{Top} + A_{s(Add'l)}\ Dia_{Add'l}) / A_{sTotal}$ =	0.625 in	b =	12.00 in	
Effective Depth, $d = T_S - C_{Vt} - d_{avg}/2$ =	5.688 in	$\rho_{PRV'D}$ =	0.01239	
$M_{U(D+LL+I)} = M_{U(D)} + M_{U(LL+I)}$ =	-0.74 k-ft/ft			
$a = A_{sTotal}\ f_y / 0.85\ f'_c\ b$ =	1.105 in			
$\phi M_n = \phi\ A_{sTotal} f_y (d - a/2)$ =	19.54 k-ft/ft	**OK! > Mu**		

Check Crack Control

(AASHTO 5.7.3.4)

Total Service $M_{D+LL+I} = M_D + M_{LL+I}$ =	0.12 k-ft/ft		
$k = [(\rho\ n)^2 + 2\ \rho\ n]^{.5} - \rho\ n$ =	0.341	j = 1 - 1/3*k =	0.886
$C = T = M_{D+LL+I} / (j\ d)$ =	0.3 k		
$f_s = T / A_{sTotal}$ =	0.33 ksi		
$f_c = 2\ C / (b\ k\ d)$ =	0.02 ksi		
Exposure Factor, γ_e =	0.75 k/in		
Depth From Extreme Tension Fiber to Closest Bar Center, $d_c = C_{Vt} + d_{avg} / 2$ =	2.81 in		
Strain Ratio, $\beta_s = 1 + d_c / [0.7 * (h-d_c)]$ =	1.706	Where h is equal to the interior slab thickness T_S	
Spacing Limit, $s_{max} = (700 * \gamma_e / \beta_s f_s) - 2 * d_c$ =	931.25 in		
Actual Spacing =	5.50 in	**OK! < Max spa**	

Part 7) Design Case 3, at Critical Section, Interior Span Side : The loads that occupy the Overhang - Strength Limit State V

Note: For this load combination, wind load is included on the noise barrier/fence.

Load Factor, γ_{LL} =	1.35	For M_{LL}	*AASHTO Table 3.4.1-1*
Load Factor, γ_{WS} =	0.40	For M_{WS}	

Dead Load Moments

Total DL Fact. $M_{U(D)} = \eta_i(\gamma_{pDC}(M_{DC1} + M_{DC2} + M_{DC3} + M_{DC4} + M_{DC5} + M_{DC5i}) + \gamma_{pDW} M_{DW})$ =	2.29 k-ft/ft	Same as Part 6) above

Live Load Moment

Factored LL + I Moment, $M_{U(LL+I)} = \eta_i\ \gamma_{LL}\ M_{(LL+I)}$ =	-2.34 k-ft/ft	Same as Part 6) above, except with new factor

Noise Barrier/Fence Wind Moment

$M_{WS} = A *\ P_W\ (N_H)\ ((N_H)/2 + B_H + T_L + T_S/2)$ =	0.00 k-ft/ft
Factored Wind Moment, $M_{U(WS)} = \eta_i\ \gamma_{WS}\ M_{WS}$ =	0.00 k-ft/ft
Total Fact. Moment = $M_{U(D)} + M_{U(LL+I)} + M_{U(WS)}$ =	-0.05 k-ft/ft

Check Moment Capacity of Reinforcement

ϕM_n =	19.54 k-ft/ft	**OK! > Mu**

Check Crack Control

(AASHTO 5.7.3.4)

Total Service Moment = $M_D + M_{LL+I} + M_{WS}$ =	0.12 k-ft/ft	
$f_s = M / A_{sTotal}\ j\ d$ =	0.33 ksi	
Exposure Factor, γ_e =	0.75 k/in	
Depth From Extreme Tension Fiber to Closest Bar Center, $d_c = C_{Vt} + d_{avg} / 2$ =	2.81 in	
Strain Ratio, $\beta_s = 1 + d_c / [0.7 * (h - d_c)]$ =	1.706	Where h is equal to the interior slab thickness T_S
Spacing Limit, $s_{max} = (700 * \gamma_e / \beta_s f_s) - 2 * d_c$ =	931.25 in	
Actual Spacing =	5.50 in	**OK! < Max spa**

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Part 8) Compute Additional Reinforcing Overhang Cutoff Length Requirement

Note: This design only necessary when additional reinforcing steel is present in the overhang.

A) Check Add'l Reinforcing Cut-Off for Design Case 1, at Critical Section, Interior Span Side : Horizontal Vehicular Collision Load - Extreme Event Limit State

Location of Critical Section with Add'l No Longer Required =	10.0 in	Distance from CL Exterior Beam
Assume the collision moment at the first interior beam, $M_{c2} = 0.4M_c$		
Interpolate to find the collision moment at the interior span side, $M_{ci} = A \times M_c$, A =	0.868	
Design Moment due to Collision Acting at Critical Section: $M_{CS} = M_{ci} L_c / (L_c + 0.8X)$		
M_{c2}=	6.7 k-ft/ft	
M_{cx}=	14.6 k-ft/ft	
L_c =	17.6 ft	
X = C + L =	2.833 ft	
M_{CS} =	12.93 k-ft/ft	
Design Axial Tensile Force acting at Critical Section:		
$P_{CS} = R_w / (L_c + 2H + 0.8X)$ =	0.71 k/ft	

Dead Load Moments

Uniform Load of OH Slab, $W_{DC1} = (T_{O\text{-}Min} + T_O) / 2 / 12 * \gamma_{conc}$ =	0.11 ksf	Load Factor, γ_{pDC} =	1.25	For M_{DC}	*AASHTO Table 3.4.1-1*
Uniform Load of Overlay, $W_{DC2} = T_L / 12 * \gamma_{conc}$ =	0.00 ksf	Load Factor, γ_{pDW} =	1.5	For M_{DW}	
Uniform Load of Interior Slab, $W_{DC1i} = T_S / 12 * \gamma_{conc}$ =	0.11 ksf	Load Modifier, η_i =	1.0	For All Loads	
Uniform Load of Sidewalk, $W_{DC5} = T_{SW} / 12 * \gamma_{conc}$ =	0.00 ksf	Resistance Factor, ϕ =	1.0	For Extreme Event Limit State	
Uniform Load of Interior Sidewalk, $W_{DC5i} = T_{SWi} / 12 * \gamma_{conc}$ =	0.00 ksf				

Moments

Exterior Slab Moment, $M_{DC1} = A\ W_{DC1}\ (O)^2 / 2$ =	0.56 k-ft/ft
Exterior Overlay Moment, $M_{DC2} = A\ W_{DC2}\ (C)^2 / 2$ =	0.00 k-ft/ft
Interior Slab Moment, $M_{DC1i} = 0.4W_{DC1i}\ (B)(L) - W_{DC1i}\ (L)^2 / 2$ =	-0.28 k-ft/ft
Interior Overlay Moment, $M_{DC2i} = 0.4W_{DC2}\ (B)(L) - W_{DC2}\ (L)_2 / 2$ =	0.00 k-ft/ft
Barrier Moment, $M_{DC3} = A\ B_W\ (B_{CG} + C)$ =	1.89 k-ft/ft
Noise Barrier/Fence Moment, $M_{DC4} = A\ N_W\ (N_{CG} + C)$ =	0.00 k-ft/ft
Sidewalk Moment, $M_{DC5} = A\ W_{DC5}\ (C)^2 / 2$ =	0.00 k-ft/ft
Interior Sidewalk Moment, $M_{DC5i} = 0.5\ W_{DC5i}\ (L_{SW} - C)(L) - W_{DC5i}\ (L)^2 / 2$ =	0.00 k-ft/ft
FWS Moment, $M_{DW} = A\ W_{DW}\ (C)^2 / 2$ =	0.10 k-ft/ft
FWS Moment, $M_{DWi} = 0.4M_{DW}\ (B)(L) - M_{DW}\ (L)^2 / 2$ =	-0.16 k-ft/ft
Total Dead Load Service Moment =	2.13 k-ft/ft
Total Fact. $M_{U(D)} = \eta_i(\gamma_{pDC}(M_{DC1}+M_{DC2}+M_{DC1i}+M_{DC2i}+M_{DC3}+M_{DC4}+M_{DC5}+M_{DC5i})+\gamma_{pDW}(M_{DW}+M_{DWi})$ =	2.65 k-ft/ft
Total Design Factored Moment, $M_u = M_{CS} + M_{U(D)}$ =	15.58 k-ft/ft

For a section under moment and axial tension, P_{CS}, the nominal resistance, M_n, may be calculated as: $\phi M_n = \phi[T(d - a/2) - P_{CS}(d/2 - a/2)]$

$A_{s(Top)}$ =	0.676 in^2/ft	$A_{s(Add'l)}$ =	0.000 in^2/ft
Avg. Dia., $d_{avg} = (A_{s(Top)}\ Dia_{Top} + A_{s(Add'l)}\ Dia_{Add'l}) / A_{sTotal}$ =	0.625 in	b =	12.00 in
Effective depth, $d = T_S + T_L - C_{Vt} - d_{avg}/2$ =	5.688 in	$\rho_{PRV'D}$ =	0.00991
$T = (A_{s(Top)} + A_{s(Add'l)})\ f_y$ =	40.58 k/ft		
Compression in concrete, $C = T - P_{CS}$ =	39.88 k/ft	Where P_{CS} is the Design Axial Tensile Force from above	
$a = C / b\ \beta_1\ f'_c$ =	0.895 in		
ϕM_n =	17.58 k-ft/ft	**OK! > Mu**	
Check that the section is not over-reinforced, AASHTO 5.7.3.3.1: $c / d = a / \beta_1 / d$ =	0.191	**OK! < 0.42**	

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LRFD Design - One Course Construction - Summary of Design Criteria and Output RIGHT OVERHANG

B) Check Add'l Steel Cut-Off for Design Case 3, at Critical Section, Interior Span Side : The loads that occupy the Overhang - Strength I

Note: For the noise barrier/fence, wind load is not included under this load combination.

Load Factor, γ_{pDC} =	1.25	For M_{DC}	*AASHTO Table 3.4.1-1*
Load Factor, γ_{pDW} =	1.5	For M_{DW}	
Load Factor, γ_{LL} =	1.75	For M_{LL}	
Load Modifier, η_i =	1.05	For All Loads	
Resistance Factor, ϕ =	0.9	For Strength Limit State	

Dead Load Moments

Uniform Load of OH Slab, $W_{DC1} = (T_{O\text{-}Min} + T_O) / 2 / 12 * \gamma_{conc}$ =	0.11 ksf
Uniform Load of Overlay, $W_{DC2} = T_L / 12 * \gamma_{conc}$ =	0.00 ksf
Uniform Load of Interior Slab, $W_{DC1i} = T_S / 12 * \gamma_{conc}$ =	0.11 ksf
Moments	
Total Dead Load Service Moment =	2.13 k-ft/ft
Total Fact. $M_{U(D)} = \eta_i(\gamma_{pDC}(M_{DC1}+M_{DC2}+M_{DC1i}+M_{DC2i}+M_{DC3}+M_{DC4}+M_{DC5}+M_{DC5i})+\gamma_{pDW}(M_{DW}+M_{DWi})$ =	2.78 k-ft/ft

Live Load Moment

Multiple Presence Factor, m =	1.20	AASHTO Table 3.6.1.1.2-1 for 1 Lane Loaded
Dynamic Load Allowance (Impact), IM =	1.33	AASHTO 3.6.2.1
M_{LL} =	-4.28 k-ft	
Equivalent Strip Width for Live Load, W = 48 + 3 S =	74.500 in	AASHTO Table 4.6.2.1.3-1 (S is in feet)
Wheel Load, P =	16.0 kips	
$M_{LL+I} = m\ (IM)\ M_{LL} / W$ =	-1.10 k-ft/ft	
Factored LL + I Moment, $M_{U(LL+I)} = \eta_i\ \gamma_{LL}\ M_{(LL+I)}$ =	-2.02 k-ft/ft	

Check Moment Capacity of Reinforcement

$A_{s(Top)}$ =	0.676 in^2/ft	$A_{s(Add'l)}$ =	0.000 in^2/ft
Avg. Dia., $d_{avg} = (A_{s(Top)}\ Dia_{Top} + A_{s(Add'l)}\ Dia_{Add'l}) / A_{sTotal}$ =	0.625 in	b =	12.00 in
Effective depth, $d = T_S + T_L - C_{Vt} - d_{avg}/2$ =	5.688 in	$\rho_{PRV'D}$ =	0.00991
$M_{U(D+LL+I)} = M_{U(D)} + M_{U(LL+I)}$ =	0.76 k-ft/ft		
$a = A_{sTotal}\ f_y / 0.85\ f'_c\ b$ =	0.884 in		
$\phi M_n = \phi\ A_{sTotal}\ f_y\ (d - a/2)$ =	15.97 k-ft/ft	**OK! > Mu**	

Check Crack Control

(AASHTO 5.7.3.4)

Total Service $M_{D+LL+I} = M_D + M_{LL+I}$ =	1.03 k-ft/ft		
$k = [(\rho\ n)^2 + 2\ \rho\ n]^{.5} - \rho\ n$ =	0.312	j = 1 - 1/3*k =	0.896
$C = T = M_{D+LL+I} / (j\ d)$ =	2.4 k		
$f_s = T / A_{sTotal}$ =	3.59 ksi		
$f_c = 2\ C / (b\ k\ d)$ =	0.23 ksi		
Exposure Factor, γ_e =	0.75 k/in		
Depth From Extreme Tension Fiber to Closest Bar Center, $d_c = C_{Vt} + d_{avg}/2$ =	2.81 in		
Strain Ratio, $\beta_s = 1 + d_c / [0.7 * (h\text{-}d_c)]$ =	1.706	Where h is equal to the interior slab thickness T_S	
Spacing Limit, $s_{max} = (700 * \gamma_e / \beta_s f_s) - 2 * d_c$ =	79.97 in		
Actual Spacing =	5.50 in	**OK! < Max spa**	

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C) Check Add'l Steel Cut-Off for Design Case 3, at Critical Section, Interior Span Side : The loads that occupy the Overhang - Strength V

Note: For this load combination, wind load is included on the noise barrier/fence.

Load Factor, γ_{LL} =	1.35	For M_{LL}	*AASHTO Table 3.4.1-1*
Load Factor, γ_{WS} =	0.40	For M_{WS}	

Dead Load Moments

Total DL Fact. $M_{U(D)} = \eta_i(\gamma_{pDC}(M_{DC1} + M_{DC2} + M_{DC3} + M_{DC4} + M_{DC5} + M_{DC5i}) + \gamma_{pDW} M_{DW})$ =	2.78 k-ft/ft	Same as Part B) above

Live Load Moment

Factored LL + I Moment, $M_{U(LL+I)} = \eta_i \gamma_{LL} M_{(LL+I)}$ =	-1.56 k-ft/ft	Same as Part B) above, except with new factor

Noise Barrier/Fence Wind Moment

$M_{WS} = A * P_W (N_H) ((N_H)/2 + B_H + T_L + T_S/2)$ =	0.00 k-ft/ft	
Factored Wind Moment, $M_{U(WS)} = \eta_i \gamma_{WS} M_{WS}$ =	0.00 k-ft/ft	
Total Fact. Moment = $M_{U(D)} + M_{U(LL+I)} + M_{U(WS)}$ =	1.23 k-ft/ft	

Check Moment Capacity of Reinforcement

ϕM_n =	15.97 k-ft/ft	**OK! > Mu**

Check Crack Control

Total Service Moment = $M_D + M_{LL+I} + M_{WS}$ =	1.03 k-ft/ft	
$f_s = M / A_{sTotal} j d$ =	3.59 ksi	
Exposure Factor, γ_e =	0.75 k/in	
Depth From Extreme Tension Fiber to Closest Bar Center, $d_c = C_{Vt} + d_{avg}/2$ =	2.81 in	
Strain Ratio, $\beta_s = 1 + d_c / [0.7 * (h - d_c)]$ =	1.706	Where h is equal to the interior slab thickness T_S
Spacing Limit, $s_{max} = (700 * \gamma_e / \beta_s f_s) - 2 * d_c$ =	79.97 in	
Actual Spacing =	5.50 in	**OK! < Max spa**

D) Results

Location of Critical Section with Additional No Longer Required =		10.0 in	
Additional Cut-off Length = Max of:	Effective Depth, d_e =	5.7 in	AASHTO LRFD 5.11.1.2
	15 d_b =	9.4 in	
	1/20 clear span =	5.3 in	
Total Length of Additional Reinforcing Beyond CL Beam =		20 in	**OK > 25 bar diameters**

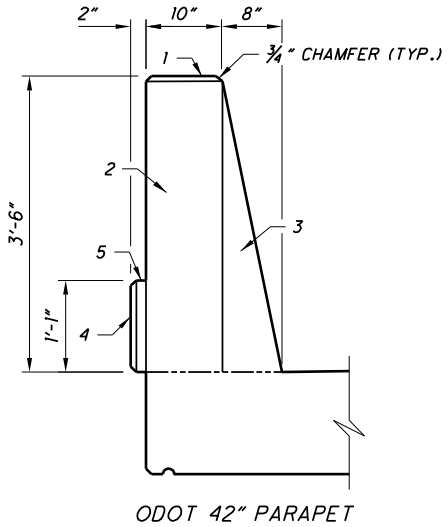
HNTB Corporation

ODOT - 42" Height Parapet

English Units

Properties in the X-Direction

	Factor	Height	Width	Area	Arm	Area*Arm
1	1	0.063	0.771	0.048	1.083	0.052
2	1	3.438	0.833	2.865	1.083	3.103
3	0.5	3.438	0.667	1.146	0.444	0.509
4	1	1.021	0.063	0.064	1.635	0.104
5	1	1.083	0.104	0.113	1.552	0.175
	Height =	3.500	ft	**4.059**	0.903	3.665
			Conc Unit Wt	0.150	kcf	
			Weight =	**0.609**	k / ft	



ODOT 42" PARAPET

ODOT - 42" Height Parapet w/ Aesthetic Treatment (same as "ODOT 42" Height Parapet, minus areas 4 and 5)

English Units

Properties in the X-Direction

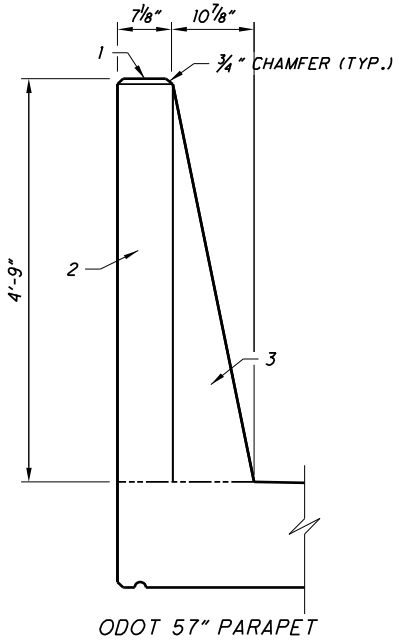
	Factor	Height	Width	Area	Arm	Area*Arm
1	1	0.063	0.771	0.048	1.083	0.052
2	1	3.438	0.833	2.865	1.083	3.103
3	0.5	3.438	0.667	1.146	0.444	0.509
	Height =	3.500	ft	**4.059**	0.903	3.665
			Conc Unit Wt	0.150	kcf	
			Weight =	**0.609**	k / ft	

ODOT - 57" Height Parapet

English Units

Properties in the X-Direction

	Factor	Height	Width	Area	Arm	Area*Arm
1	1	0.063	0.531	0.033	1.203	0.040
2	1	4.688	0.594	2.783	1.203	3.349
3	0.5	4.688	0.906	2.124	0.604	1.283
	Height =	4.750	ft	**4.940**	0.946	4.672
			Conc Unit Wt	0.150	kcf	
			Weight =	**0.741**	k / ft	



ODOT 57" PARAPET

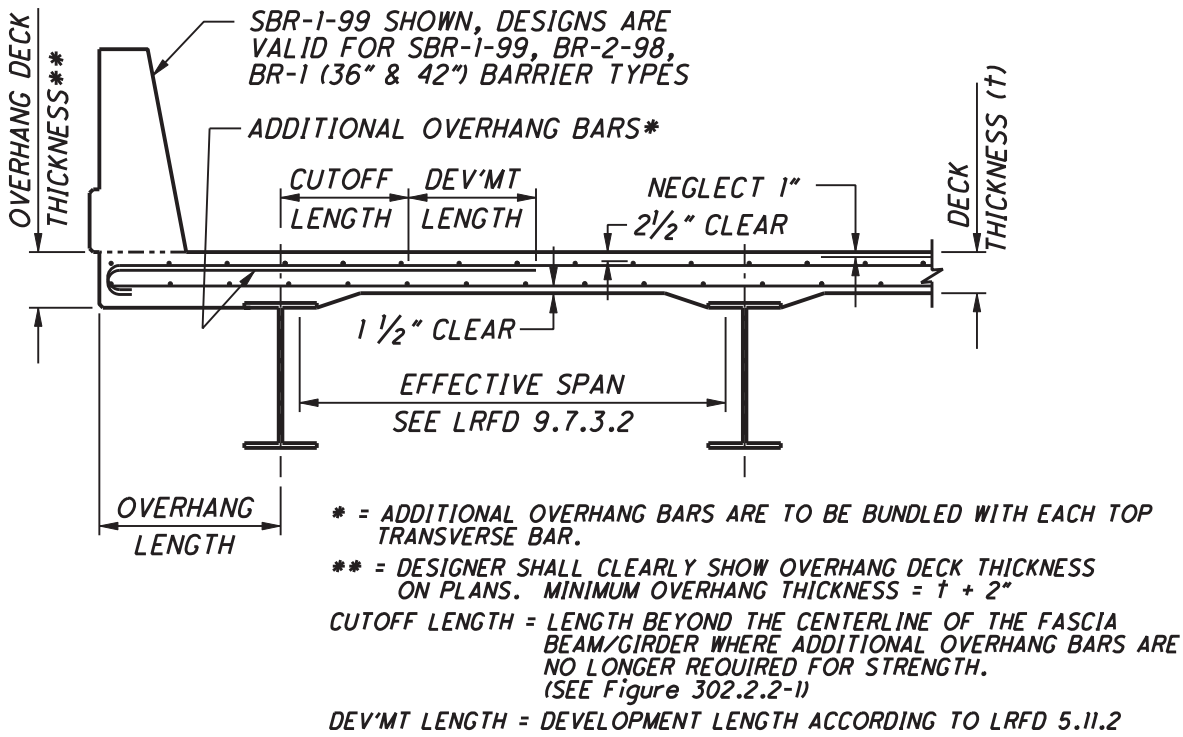
Concrete Deck Design Aid

Effective Span Length (ft.)	Deck Thickness (in.)	Overhang Deck Thickness (in.)	Transverse Steel						Longitudinal Steel			
			Top Bars				Bottom Bars		Top Bars		Bottom Bars	
			Size	Spa. (in.)	Additional Overhang Bar Size	Cutoff Length (in.)	Size	Spa.	Size	Spa.	Size	Spa.
7.0	8.50	10.50	#5	6.50	#5	50.0	#5	6.50	#4	12.50	#5	13.00
7.5	8.50	10.50	#5	6.25	#5	50.0	#5	6.25	#4	12.00	#5	12.25
8.0	8.50	10.50	#5	6.00	#5	50.0	#5	6.00	#4	11.50	#5	11.50
8.5	8.75	10.75	#5	6.00	#4	50.0	#5	6.00	#4	11.50	#5	11.25
9.0	8.75	10.75	#5	5.75	#4	50.0	#5	5.75	#4	11.00	#5	10.75
9.5	9.00	11.00	#5	5.75	#4	50.0	#5	5.75	#4	11.00	#5	10.50
10.0	9.25	11.25	#5	5.50	#4	50.0	#5	5.50	#4	10.50	#5	10.50
10.5	9.25	11.25	#5	5.25	#4	50.0	#5	5.25	#4	10.00	#5	10.00
11.0	9.50	11.50	#5	5.00	#4	38.0	#5	5.00	#4	9.50	#5	10.00
11.5	9.75	11.75	#5	5.00	#4	38.0	#5	5.00	#4	9.50	#5	9.75
12.0	9.75	11.75	#6	6.00	#4	38.0	#5	6.00	#4	8.00	#5	9.50
12.5	10.00	12.00	#6	6.00	#4	21.0	#5	6.00	#4	8.00	#5	9.25
13.0	10.25	12.25	#6	6.00	#4	21.0	#5	6.00	#4	8.00	#5	9.25
13.5	10.25	12.25	#6	5.75	#4	21.0	#5	5.75	#4	7.75	#5	9.00
14.0	10.50	12.50	#6	5.75	#4	21.0	#5	5.75	#4	7.75	#5	9.00

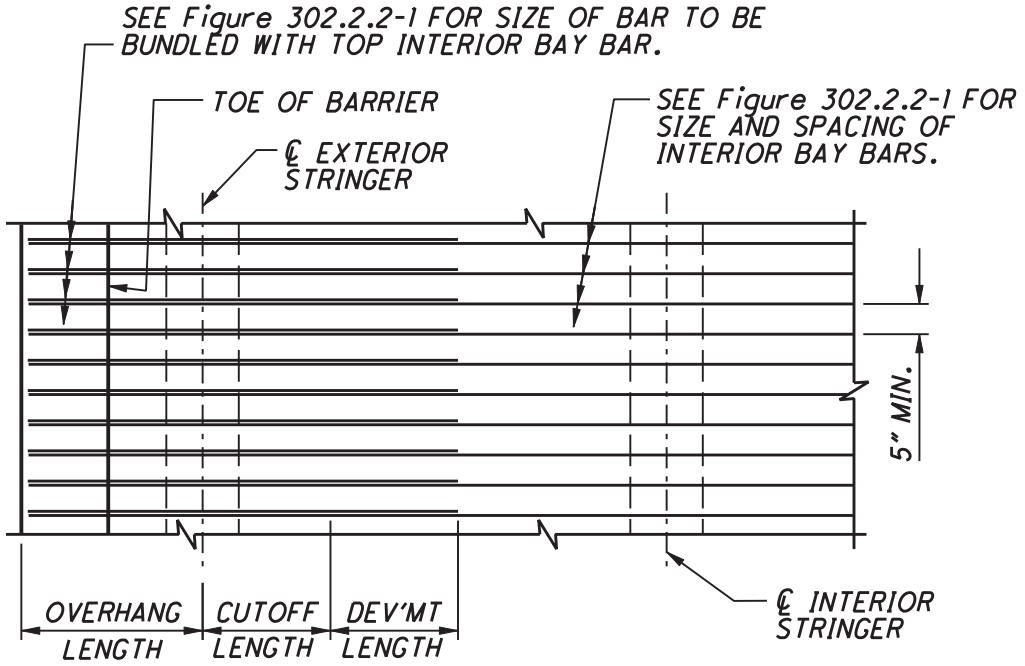
Notes:

1. Designs in accordance with AASHTO LRFD Bridge Design Specifications and the 2007 ODOT Bridge Design Manual
2. Design Assumptions:
   a. Four or more beam/girder lines
   b. Transverse steel is placed perpendicular to beam/girder lines
   c. Normal weight concrete with f'c = 4.5 ksi
   d. Reinforcing steel with fy = 60 ksi
   e. Monolithic Wearing Surface = 1.0 in.
   f. Future Wearing Surface = 0.06 ksf
   g. *LRFD 5.7.3.4* - Exposure Factor (γ_e) = 0.75
   h. Top cover = 2.50 in.; Bottom cover = 1.50 in.
   i. Maximum overhang width = 4.0 ft. (measured from cl. of fascia beam/girder to deck edge)
   j. Valid for BR-1 (36" & 42"), SBR-1-99, BR-2-98, and TST-1-99 barrier systems
3. Calculate Effective Span Length according to *LRFD 9.7.3.2* and round up to the nearest 0.5 ft. increment
4. Minimum Deck Thickness in accordance with BDM Section 302.2.1
5. Cutoff Length = length beyond the centerline of the fascia beam/girder where additional overhang bars are no longer required for strength.
6. Longitudinal bar spacing does not include additional reinforcing required for negative moments in accordance with *LRFD 5.7.3.2* (for prestressed beams) and *LRFD 6.10.1.7* (for steel beams/girders)
7. Refer to Figure 302.2.2-2 and Figure 302.2.2-3 for more information

Figure 302.2.2-1



DECK SECTION



OVERHANG PLAN VIEW

TYPICAL DECK DETAILS-- ODOT LRFD STANDARD DECK DESIGN

Figure 302.2.2-2

Calculations For	I-90 Cleveland InnerBelt Bridge 6	Job No.	49633	Sheet No.
Made by	CDD	Date	2/18/2011	
Checked by	AKS	Date	5/26/2011	
Backchecked by	CDD	Date	5/26/2011	

HNTB

Title: E 14th ST ON-RAMP TO I-90 WB AND FUTURE (CCG3) MAINLINE I-90 WB OVER E 9th ST

NOTE: Input data is denoted by shading. All other values are calculations performed by the spreadsheet.

Slab Area =	11786.9	ft^2	(Measured in Microstation)
Nominal Slab thickness =	8.50	in.	
Slab Volume = V_s =	8349.1	ft^3	(not including haunch)
Haunch thickness = h =	3.83	in.	(Average 4 1/2" at Rear Abut., 2"at Midspan, 5" at Fwd. Abut.)
Top Beam width = b_f =	48.00	in.	
66" Concrete Beam (B1 thru B3) = L_1 =	139.04	ft / beam	
66" Concrete Beam (B4 thru B10) = L_2 =	139.74	ft / beam	
Number of Beam Lines(B1 thru B3) = n_1 =	3		
Number of Beam Lines(B4 thru B10) = n_2 =	7		
Beam(B1 thru B3) Haunch Volume = V_{h1} =	532.99	ft^3	$=h*b_f*L_1*n_1$
Beam(B4 thru B10) Haunch Volume = V_{h2} =	1249.89	ft^3	$=h*b_f*L_2*n_2$
Total Haunch Volume = V_h =	1782.9	ft^3	$=V_{h1} + V_{h2}$
Total Deck Volume =	375.3	cu. yd.	$=V_h + V_s$

PRESTRESSED BEAM DESIGN

Calculations For	Innerbelt Bridge No. 6	Job No.	49633	Sheet No.	
Made by	AKS	Date	4/18/2011	4/18/2011	1/10/2012
Checked by	DBT	Date	4/18/2011	4/18/2011	1/16/2012
Backchecked by	AKS	Date	4/19/2011	4/19/2011	1/16/2012

Title: E 14th Street On-Ramp to I-90 WB and Future I-90 WB over E 9th Street (Bridge No. 6, CUY-90-1627)

BEAM DESIGN APPROACH

Beams Considered: 66" Modified AASHTO Type 4

Strand Arrangements Considered: Draped vs. Debonded
Section 302.5.2.2 of the ODM indicates that while straight parallel strand with no debonding is preferred, both debonded and draped patterns (or a combination there of) are allowed. The hold down point for draped strands shall be at least 5' from the midspan.

Dead and live loads are per the ODM and Project Design Criteria.

FWS = 60 psf

Transformed sections are not permissable ODM 302.5.2.2.

Intermediate diaphragms are required at quater points for spans exceeding 80' per ODOT Prestressed Concrete I-Beam Bridge Details. ODOT details allow either concrete or steel diaphragms. For the final design, it was assumed that the diaphragms are steel.

Hold down forces shall not exceed limits defined in ODM Section 302.5.2.2.e.

Calculations For	Innerbelt Bridge No. 6	Job No.	49633	Sheet No.	
Made by	AKS	Date	4/18/2011	4/18/2011	1/10/2012
Checked by	DBT	Date	4/18/2011	4/18/2011	1/16/2012
Backchecked by	AKS	Date	4/19/2011	4/19/2011	1/16/2012

HNTB

Title: E 14th Street On-Ramp to I-90 WB and Future I-90 WB over E 9th Street (Bridge No. 6, CUY-90-1627)

CONSPAN Beam Design Input

Materials:

Deck Concrete:	f'_c =	4500 psi		Note: Project Design Criteria
Beam Concrete:	f'_{ci} =	7500 ksi	at Release	Note: ODOT BDM 205.5, 5ksi & 7 ksi
	f'_c =	11000 ksi	at 28 Day	Allowed per coordination with precast supplier
Prestressing Steel:	Diameter =	0.60 in		
	Area =	0.217 in^2		
	Type =	270K Low Relaxation		
Reinforcing Steel:	ASTM A615 Grade 60			

Bridge Geometry Data:

Bridge Overall Width:	Out-to-Out Coping	85.9 ft	Max. Actual	Note: Actual Bridge Width Varies,
Barriers:	Left	1.5 ft		see below
	Right	1.5 ft		
Clear Roadway:		82.9		
Skew:	Left	12.1 deg		Note: Skew is relative to Beam CL
Beam	Trial Depth	66.0 in		
	Trial Top Flange Width	48.0 in		Revised from ODOT Std. 36"
Span "1" Beam:	Plan Span Length	139.736 ft		Note: Measured along Baseline
	Precast Length =	139.736 ft		
	Brg to Brg Length =	138.070 ft		
	Pier CL / Precast Offset =	-0.833 ft		
No. of Lanes:	Clear Roadway/12 =	6.908	Use 7 Lanes	
Slab Thickness:	+ 1" wearing surface =	8.50 in		
Beam Spacing:	9 Spaces @	8.833 ft		

		Average	Maximum	
Left & Right Overhang:	Left	3.65 ft	3.80 ft	Note: Actual Overhangs Vary, for Average Overhang see attached sketch
	Right	3.17 ft	3.54 ft	
Overhang Checks:	0.45 * beam spacing	OK	3.98 ft	
	0.85 * beam depth	OK	4.68 ft	
	max	OK	5.00 ft	
	min for OS drain (1.75 + 0.5*flange)	OK	3.75 ft	

Note: One of the beam spaces varies. For the design run, maximum spacing and average overhangs were used, resulting in 3.65' + 9 x 8.833' + 3.17' = 86.32'

Calculations For	Innerbelt Bridge No. 6	Job No.	49633	Sheet No.	
Made by	AKS	Date	4/18/2011	4/18/2011	1/10/2012
Checked by	DBT	Date	4/18/2011	4/18/2011	1/16/2012
Backchecked by	AKS	Date	4/19/2011	4/19/2011	1/16/2012

HNTB

Title: E 14th Street On-Ramp to I-90 WB and Future I-90 WB over E 9th Street (Bridge No. 6, CUY-90-1627)

CONSPAN Beam Design Input

Loads:

Dead Load

Slab:	Effective Depth	7.50 in		
Beam Type:	Modified AASHTO Type IV, 72 in.	66 inch		
Stay-in-Place Forms:	Weight	0.000 ksf		
	Exterior Beam Load -	0.000 k/ft		
	Interior Beam Load -	0.000 k/ft		
Integral Wearing Surface:	Depth	1.00 in		
	Exterior Beam Load -	0.103 k/ft		Note: Load Calculated Using Max. Overhang
	Interior Beam Load -	0.110 k/ft		
Haunch:		0.150 k/ft		

Slab Dead Load to Outside Beams (AASHTO C4.6.2.2.1)

Load calculated internally by CONSPAN (tributary width per CONSPAN User Manual, page TH-29)

$SW_{(slab)}$ = 0.7083ft x (3.7968ft + 4.4167ft) x 0.150 kcf = 0.873 k/ft Note: Load Calculated Using Max. Overhang

Load calculated by lever rule per AASHTO LRFD C4.6.2.2.1

$SW_{(slab)}$ = [(0.7083ft x 12.6301ft x 0.150 kcf) x 6.3151ft] / 8.8333ft = 0.959 k/ft Note: Load Calculated Using Max. Overhang

Net Additonal Slab Load = 0.087 k/ft <-- Apply as Precast DC Line Load to Ext. Beams

Diaphragms	Steel assumed	Interior	0.55 k	
		Exterior	0.27 k	

Composite Load

Left Barrier Weight:	0.612 k/ft	
Right Barrier Weight:	0.746 k/ft	
Future Wearing Surface:	0.060 ksf	over Clear Roadway width

Live Load: AASHTO LRFD HL-93 Loading

Restraining Moments: PCA Method w/ continuity established at 28 days

Operational Importance Factor (AASHTO 1.3.5): 1

Effective Slab Width (AASHTO LRFD 4.6.2.6):

SPAN 1

Interior Beam:

Smaller of:	Span Length / 4 =	419.21 in	
	12 x slab depth + top flange width /2 =	114.00 in	
	Avg. Support Spacing =	106.00 in	<-- Controls

Exterior Beam:

Smaller of:	106/2 + Span Length / 8 =	262.60 in		
	106/2 + 6 x slab depth + top flange width /	110.00 in		
	106/2 + Overhang Width =	98.56 in	<-- Controls	Note: Width Calculated Using Max. Overhang

Calculations For	Innerbelt Bridge No. 6	Job No.	49633	Sheet No.
Made by	AKS	Date	1/27/2011	4/18/2011
Checked by	DBT	Date	2/2/2011	4/18/2011
Backchecked by	AKS	Date	2/3/2011	4/19/2011

HNTB

Title: E 14th Street On-Ramp to I-90 WB and Future I-90 WB over E 9th Street (Bridge No. 6, CUY-90-1627)

	STATION (feet)	ELEVATION (feet)	GRADE (%)	VC LENGTH (feet)
PVC	169+50.00	694.833	1.790	
PVI	172+75.00	700.650		650
PVT	176+00.00	697.400	-1.000	

LOCATION	STATION (feet)	PG ELEVATION (feet)	GRADE (%)	Avg. Grade (%)
Span 1 Grade Check				
Rear Abutment	175+59.33	697.771	-0.825	-0.91 %
Forward Abutment	a 177+09.27	696.307	-1.000	

Span 1 Precast Length =	139.736 ft
Slope (deg) =	-0.5229332
Adjusted Precast Length =	139.742 ft
Additional Length =	0.07 in - No Impact On Precast Length

	STATION (feet)	ELEVATION (feet)	GRADE (%)	VC LENGTH (feet)
PVC	176+53.09	696.880	-1.000	
PVI	a 177+55.00	695.980		180
PVT	a 178+45.00	695.710	-0.300	

LOCATION	STATION (feet)	PG ELEVATION (feet)	GRADE (%)	Avg. Grade (%)
Span 1 Grade Check				
Rear Abutment	175+59.33	697.937	-1.000	-0.89 %
Forward Abutment	a 177+09.27	696.380	-0.782	

Span 1 Precast Length =	139.736 ft
Slope (deg) =	-0.510355
Adjusted Precast Length =	139.742 ft
Additional Length =	0.07 in - No Impact On Precast Length

Calculations For	Innerbelt Bridge No. 6	Job No.	49633	Sheet No.
Made by	AKS	Date	1/27/2011	4/18/2011
Checked by	DBT	Date	2/2/2011	4/18/2011
Backchecked by	AKS	Date	2/3/2011	4/19/2011

Title: E 14th Street On-Ramp to I-90 WB and Future I-90 WB over E 9th Street (Bridge No. 6, CUY-90-1627)

Strand Hold-Down Force Calculation

Span "1"

Beam Depth =	66.0 in	Total strands held down =	15	per web
End Strand Elevation "in"	59.00 in	End of beam to hold-down =	754.6 in	= 0.45 x 139.74 x 12
Middle Strand Elevation "in"	7.00 in	f_s =	270.0 ksi	
Strand area =	0.217 in^2	0.75 * f_s * strand area =	43.94 kips	

h =	52.0 in		
Resultant =	756.4 in		
Hold-down per strand =	**3.02 kips**	**OK**	(Allowable = 4 kips, ODOT BDM 302.5.2.2e)
Total Hold-down =	**45.31 kips**	**OK**	(Allowable = 48 kips, ODOT BDM 302.5.2.2e)

Calculations For	Innerbelt Bridge No. 6	Job No.	49633		Sheet No.
Made by	AKS	Date	1/27/2011	4/18/2011	1/10/2012
Checked by	DBT	Date	2/2/2011	4/18/2011	1/16/2012
Backchecked by	AKS	Date	2/3/2011	4/19/2011	1/16/2012

Title: E 14th Street On-Ramp to I-90 WB and Future I-90 WB over E 9th Street (Bridge No. 6, CUY-90-1627)

Check Equivalent Static Loads for Transport Forces (PCI Design Handbook, Table 5.2.1) - Span 1

$f'c$ =	11000 psi	
Max. Compressive Stress, 0.6 $f'c$ =	6.600 ksi	
Max. Tensile Stress, 5 $(f'c)^{1/2}$ =	-0.524 ksi	(PCI Design Handbook, 5.2.4)

Release Stresses from CONSPAN - Span 1 (Beam 1 Controlling)

@ Midspan

Prestress		Beam-Self	
Precast-top	-1.371 ksi	Precast-top	1.793 ksi
Bottom	6.269 ksi	Bottom	-1.815 ksi

Beams Supported at Bearings during Transport (Checked at Midspan)

Top of Precast

σ_{max} =	1.793 + 1.793/2 +-1.371 =	1.319 ksi	OK
σ_{min} =	1.793 - 1.793/2 +-1.371 =	-0.475 ksi	OK

Bottom of Precast

σ_{max} =	-1.815 + -1.815/2 +6.269 =	3.547 ksi	OK
σ_{min} =	-1.815 - -1.815/2 +6.269 =	5.362 ksi	OK

Calculations For	Innerbelt Bridge No. 6	Job No.	49633		Sheet No.
Made by	AKS	Date	8/1/2011	1/10/2012	
Checked by	CAB	Date	8/2/2011	1/16/2012	
Backchecked by	AKS	Date	8/2/2011	1/16/2012	

HNTB

Title: E 14th Street On-Ramp to I-90 WB and Future I-90 WB over E 9th Street (Bridge No. 6, CUY-90-1627)

Check Anchorage Zone Reinforcement (AASHTO LRFD 5.10.10) - 66" Modified AASTHO Type IV

Notes:

1) Stirrups between the end of beam and centerline bearing are assumed to have negligible contribution to vertical shear with no effective deduction in area provided.

2) CONSPAN determines the area of bursting reinforcement based on the total number of stands. With the utilization of draped strands and/or other strands placed at the top of the beam at the end, the bursting force is effectively distributed into multiple zones at the beam end. Therefore the area of bursting reinforcement will be adjusted based on the percentage of strands for a particular zone.

From CONSPAN Anchorage Zone Reinforcement Output

Beam:	1	Span:	1	
	F_{pi}	=		2636.55 kips
	f_s	=		20.0 ksi
	h/4	=		16.50 in
	$A_{brst,total}$	=		5.27 in^2
	Total No. of Strands	=		58

At End Bent Beam End

Anchorage Zone 1 - Top of Beam, Above Dapped End in Flange

Total Strands @ Top	=		15
Percentage of Total Strands	=		25.9%
$A_{brst,1}$	=		1.36 in^2

Area Provided - Zone 1

Vert. Steel	Bar Size	Bar Area	No. of Legs	Total Area	
401	#4	0.20 in^2	8	1.60 in^2	
				1.60 in^2	OK

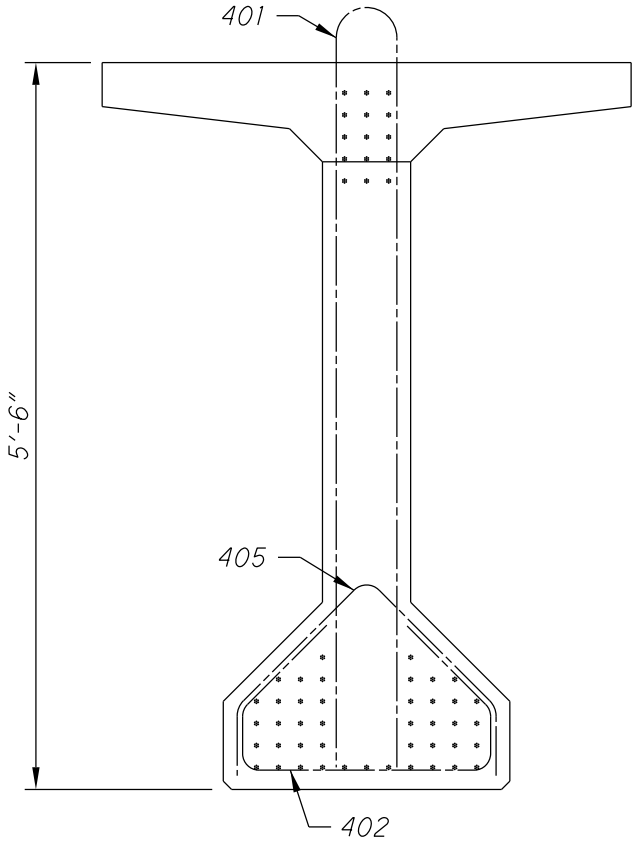
Anchorage Zone 2 - Bottom of Beam

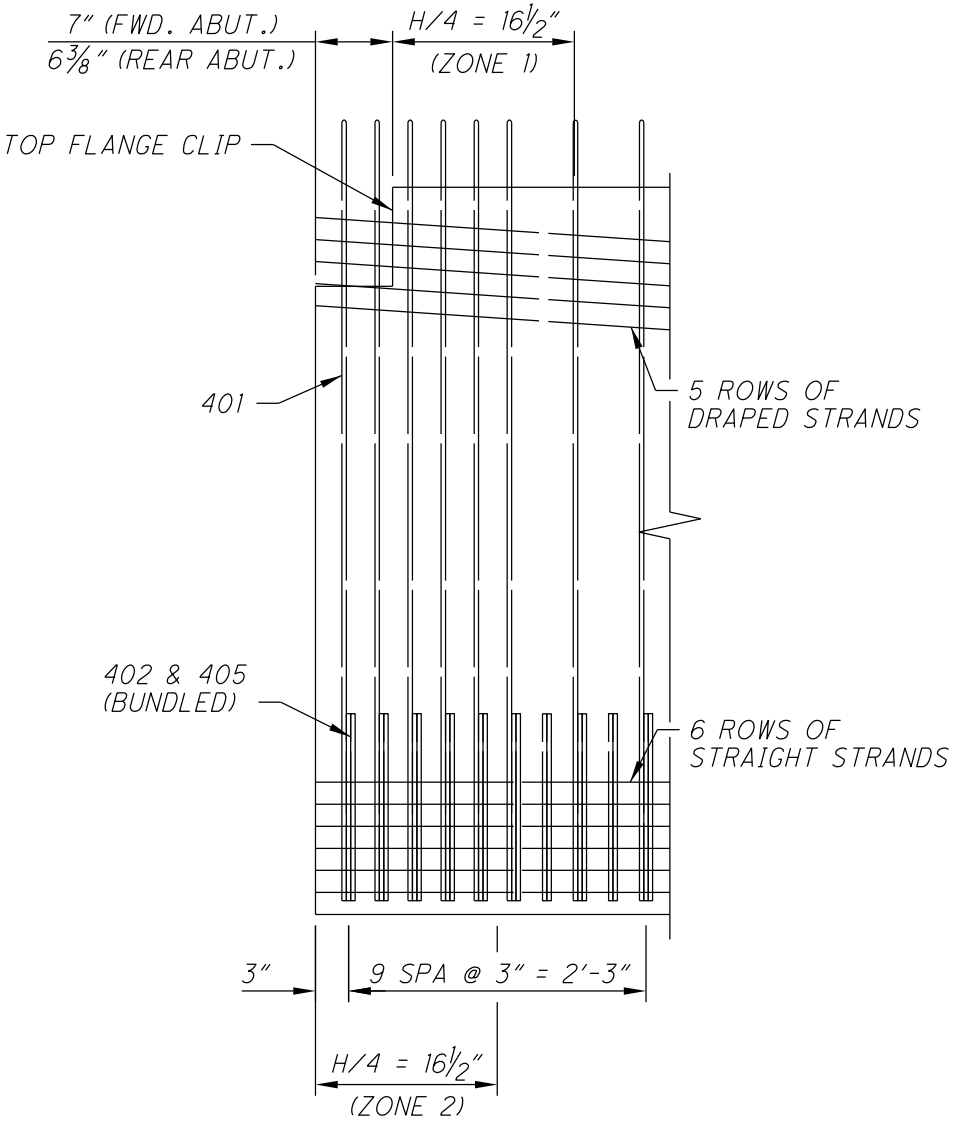
Total Strands @ Bot.	=		43
Percentage of Total Strands	=		74.1%
$A_{brst,2}$	=		3.91 in^2

Area Provided - Zone 2

Vert. Steel	Bar Size	Bar Area	No. of Legs	Total Area	
401	#4	0.20 in^2	10	2.00 in^2	
402 & 405	#4	0.20 in^2	10	2.00 in^2	(say 402 and 405 bars act as a single stirrup)
				4.00 in^2	OK

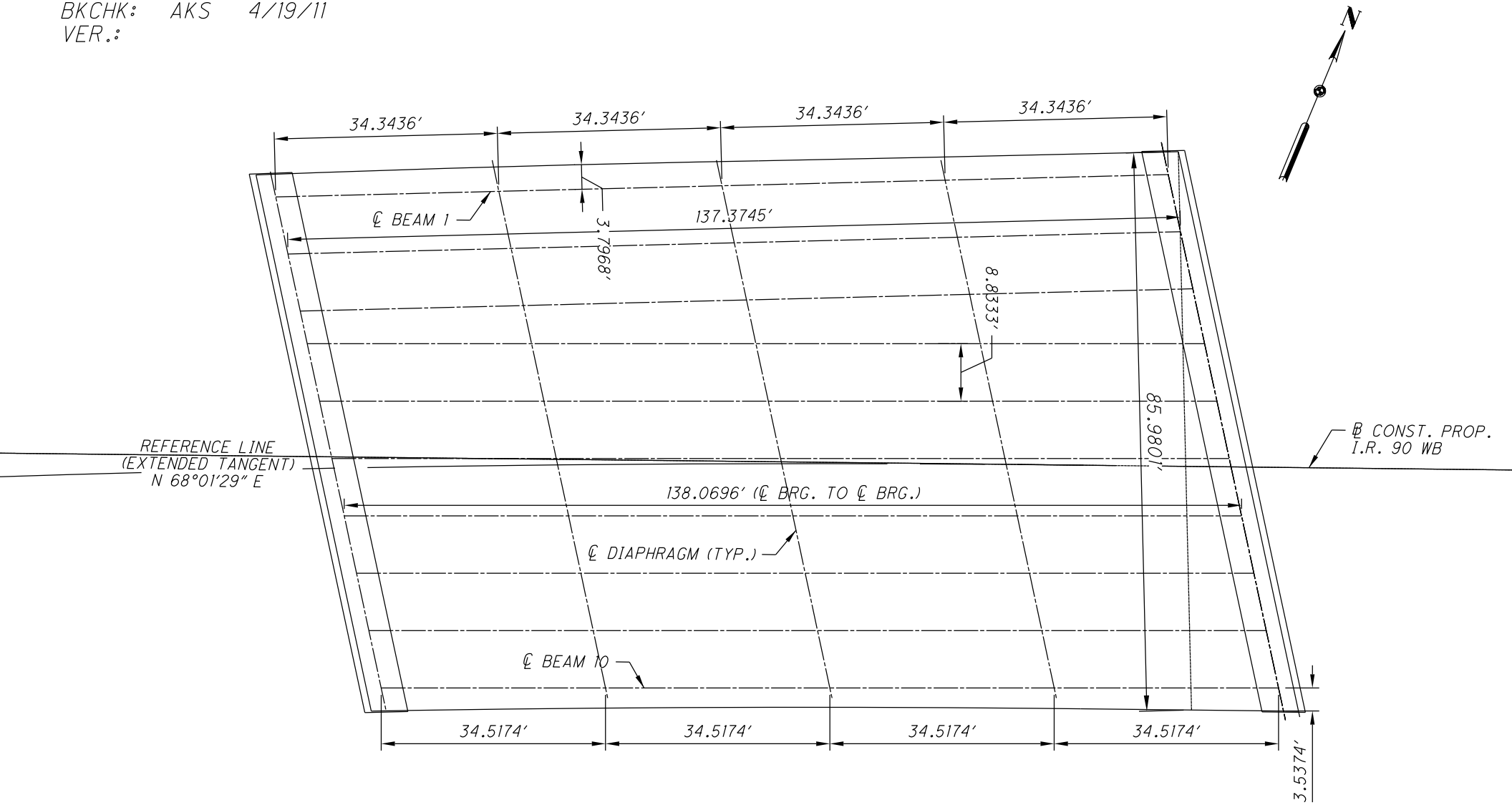
BRIDGE 6 - ANCHORAGE ZONE REINFORCING





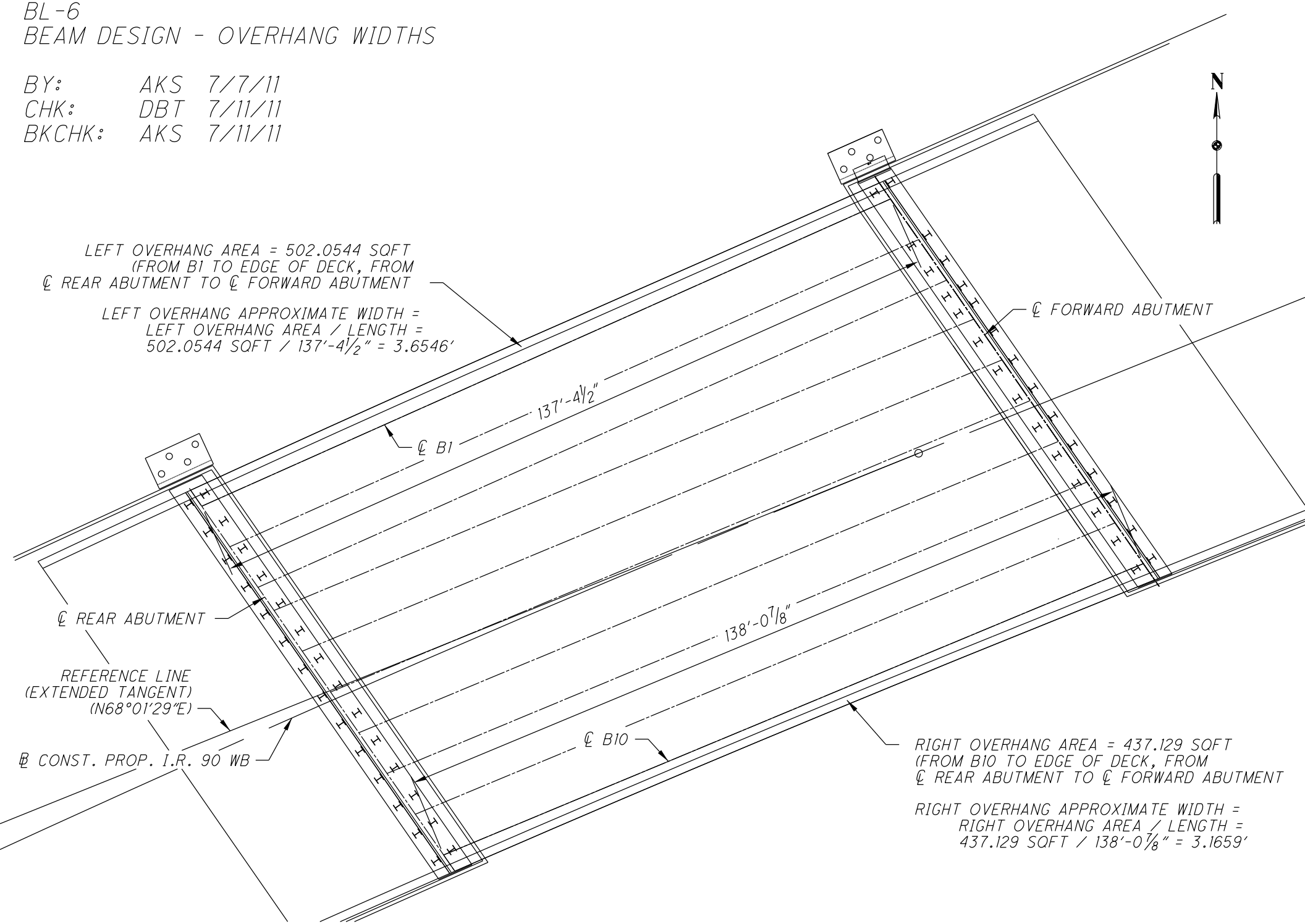
PRESTRESS BEAM DESIGN

BY:	AKS	4/18/11
CHK:	DBT	4/18/11
BKCHK:	AKS	4/19/11
VER.:		



BL-6
BEAM DESIGN - OVERHANG WIDTHS

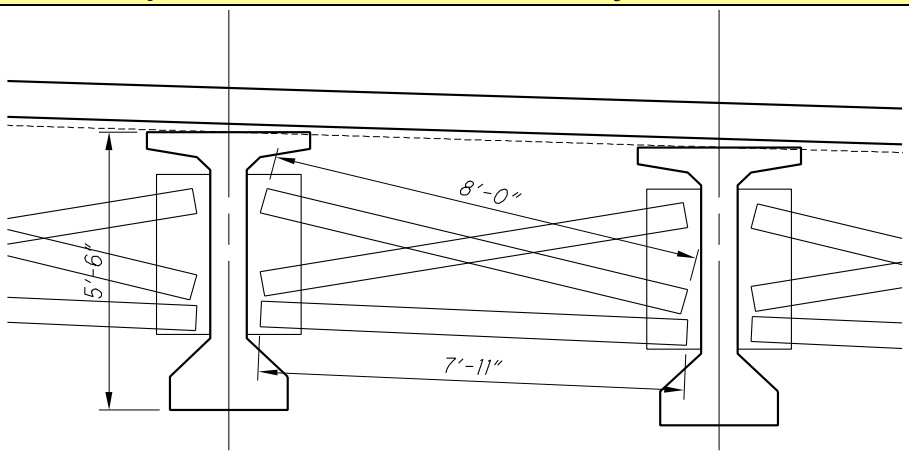
BY: AKS 7/7/11
CHK: DBT 7/11/11
BKCHK: AKS 7/11/11



Calculations For	Innerbelt Bridge No. 6	Job No.	49633	Sheet No.
Made by	AKS	Date	4/18/2011	
Checked by	DBT	Date	4/18/2011	
Backchecked by	AKS	Date	4/19/2011	

HNTB

Title: E 14th Street On-Ramp to I-90 WB and Future I-90 WB over E 9th Street (Bridge No. 6, CUY-90-1627)



Bottom Chord

Shape =	6x6x3/8	
Weight =	14.9 lb/ft	
Length =	7.92 ft	(measured in Microstation)
Bottom Chord Weight =	117.96 lb	

"x" Chords

Shape =	6x4x5/16	
Weight =	10.3 lb/ft	
Length =	8.00 ft	(measured in Microstation)
Total No. of Members =	2	
"x" Chords Weight =	164.80 lb	

Bent Plate

Weight =	490.0 lb/ft3
Height =	3.2 ft
Width =	1.5 ft
Thickness =	0.5 in.
Total Plate Volume =	0.2 ft3
Bent Plate Weight =	96.98 lb

Interior Beam Diaphragm

Bottom Chord + "x" Chords + (2*Bent Plate) + 15% for Misc. Steel = 0.55 kip

Exterior Beam Diaphram

Interior Beam Diaphragm / 2 = 0.27 kip

Calculations For	Innerbelt, Bridge 6	Job No.	49633	Sheet No.
Made by	AKS	Date	4/18/2011	
Checked by	DBT	Date	4/18/2011	
Backchecked by	AKS	Date	4/19/2011	
Revised	DBT		9/9/11	

HNTB

PROPERTIES OF SECTION DESCRIBED BY NODES COORDINATES

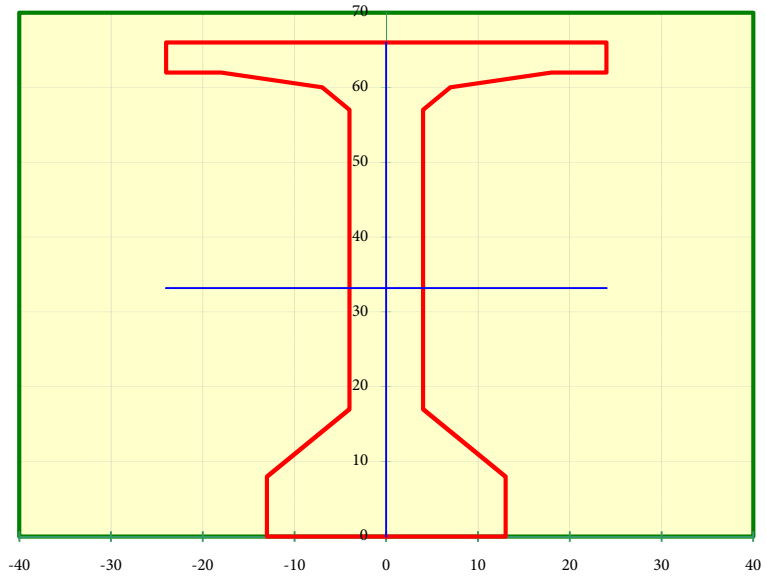
Units: in

REVISED BEAM SHAPE REQUESTED BY CONTRACTOR

Nodes Coordinates		
	X	Y
1	0.000	0.000
2	-13.000	0.000
3	-13.000	8.000
4	-4.000	17.000
5	-4.000	57.000
6	-7.000	60.000
7	-18.000	62.000
8	-24.000	62.000
9	-24.000	66.000
10	24.000	66.000
11	24.000	62.000
12	18.000	62.000
13	7.000	60.000
14	4.000	57.000
15	4.000	17.000
16	13.000	8.000
17	13.000	0.000
18	0.000	0.000
19		
20		
21		
22		
23		
24		
25		
26		
27		
28		
29		
30		
31		
32		
33		
34		
35		
36		
37		
38		

Section Properties About Centroidal Axis Parallel to Original Axis			
f =	0.000 deg	X-dim =	48 in
A =	956.00 in^2	Y-dim =	66 in
Xcg =	0.000 in	Sx(top) =	16471.8 in^3
Ycg =	33.204 in	Sx(bot) =	16269.1 in^3
Ixo =	540204.4 in^4	Sy(left) =	2436.3 in^3
Iyo =	58471.3 in^4	Sy(right)=	2436.3 in^3
Ixyo =	0.0 in^4		

Section Properties About Principal Axis			
f =	0.000 deg	Sxp(top) =	16471.8 in^3
Ixp =	540204.4 in^4	Sxp(bot) =	16269.1 in^3
Iyp =	58471.3 in^4	Syp(left) =	2436.3 in^3
Ixyp =	0.0 in^4	Syp(right)=	2436.3 in^3
J =	598675.8 in^4		

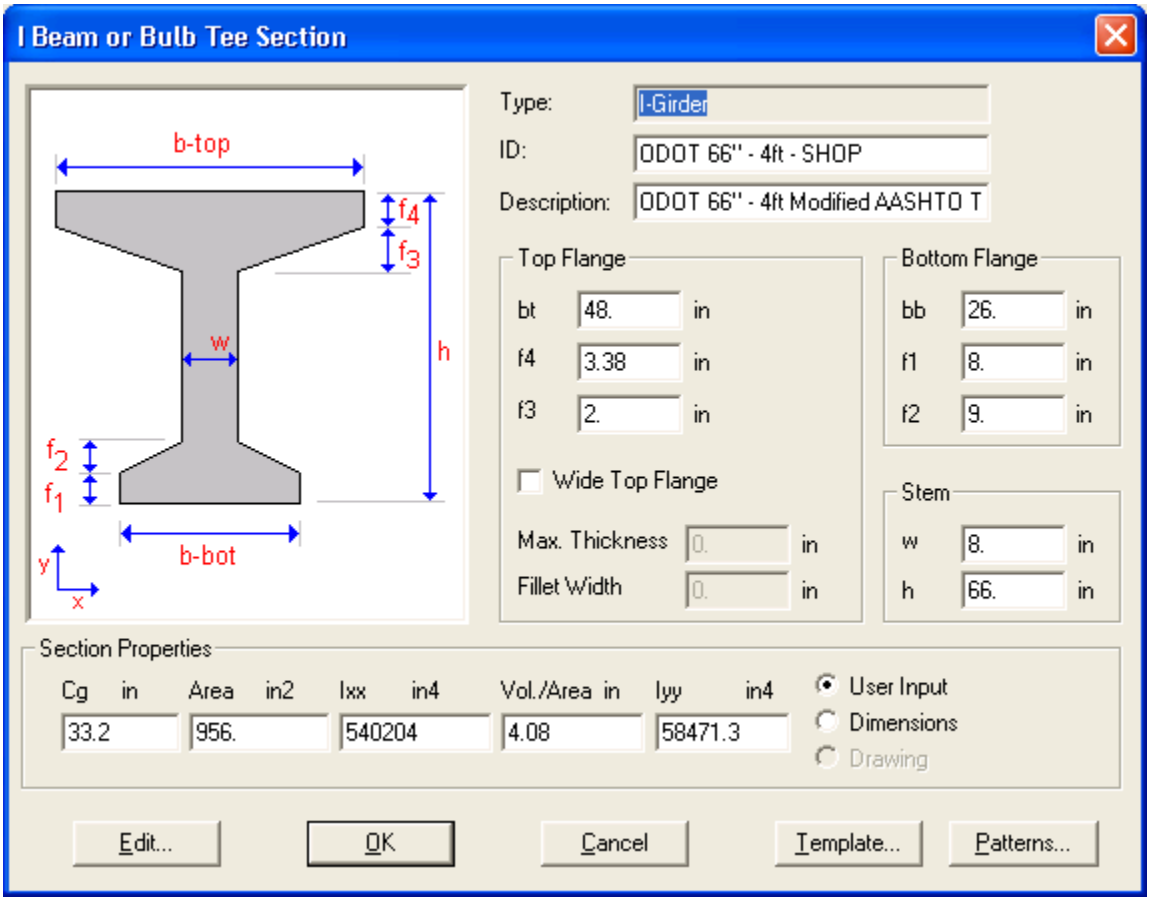


Perimeter = 234.3 in.

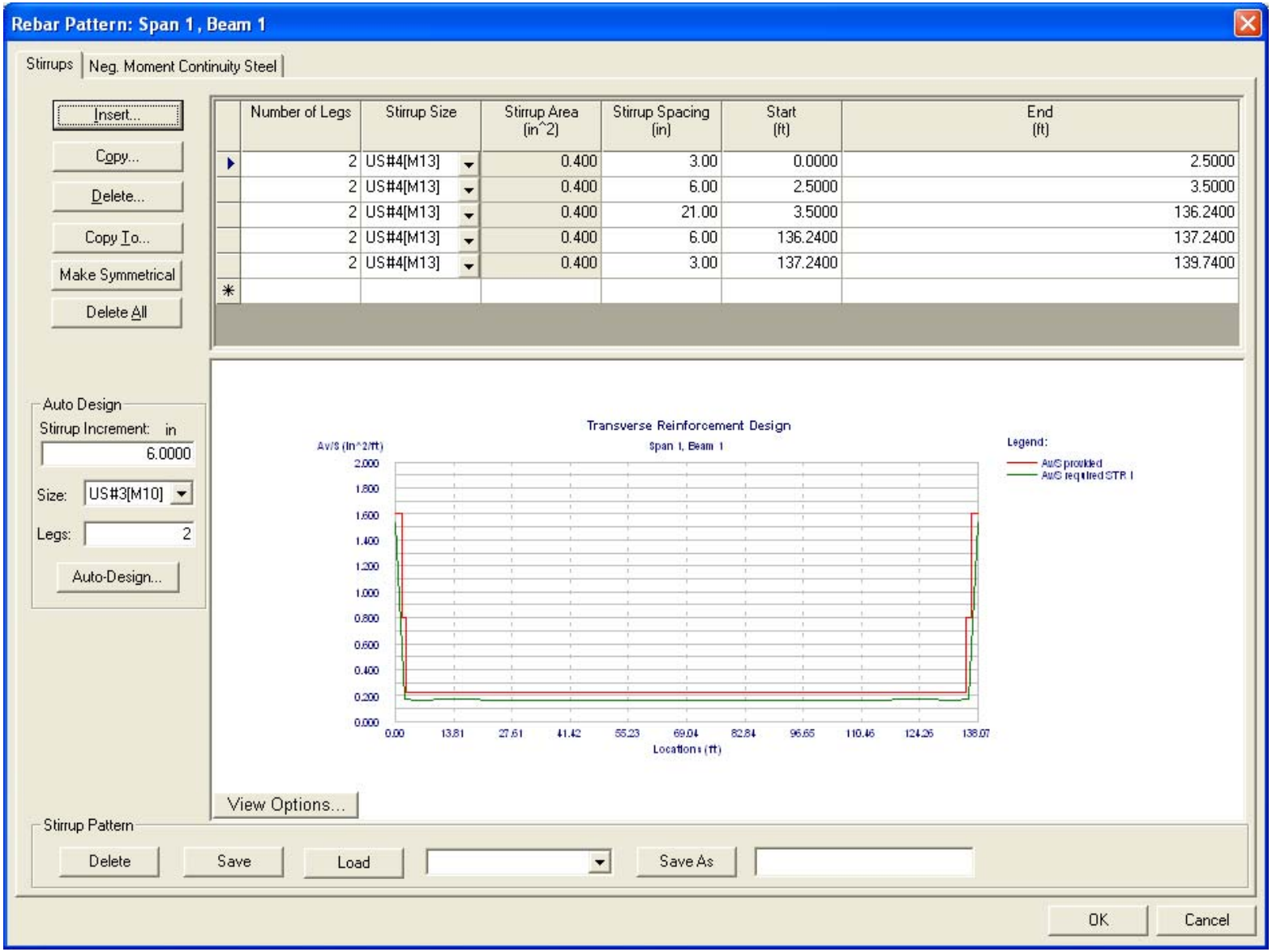
Vol / Area = 4.080 in.

(See Conspan User Manual, Vol / Area = Cross Sectional Area / Perimeter)

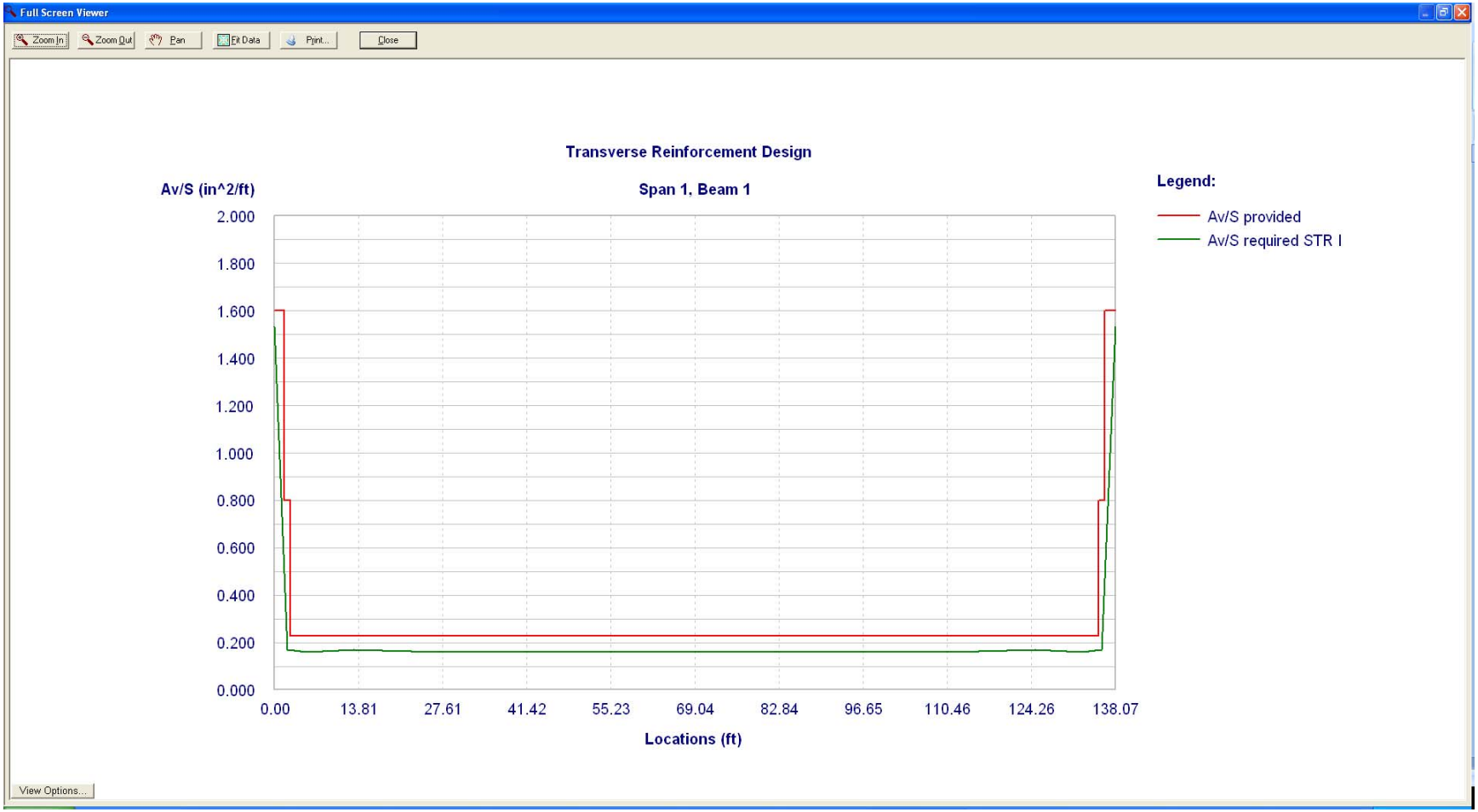
REVISED SHAPE REQUESTED BY CONTRACTOR

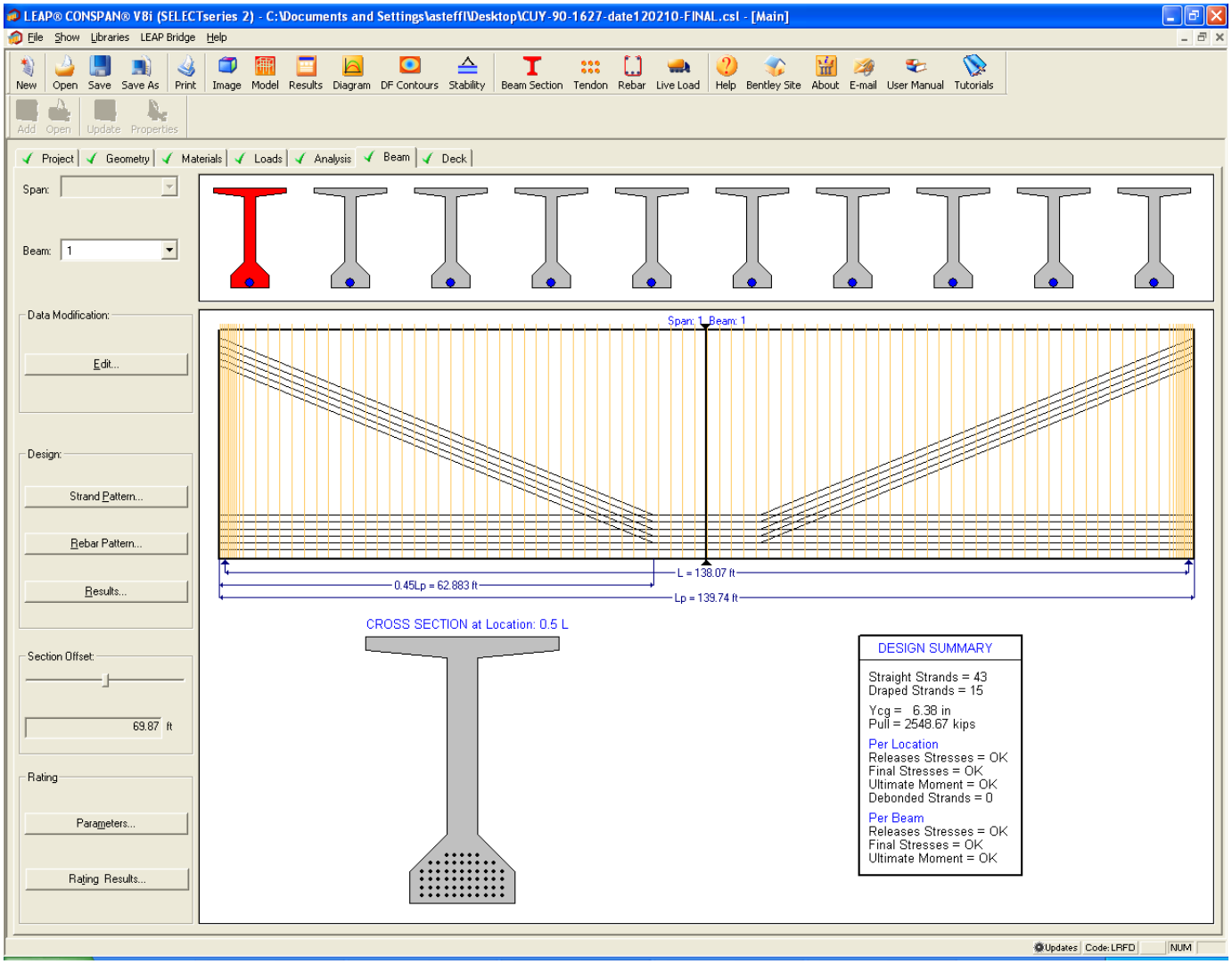


Conspan Beam Library, Beam Section Properties



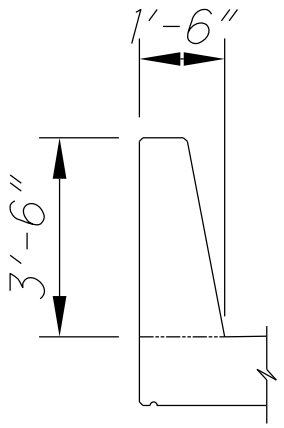
Rebar Spacing


Full Screen Viewer
Zoom In
Zoom Out
Pan
Fit Data
Print...
Close
Transverse Reinforcement Design
Span 1, Beam 1
Av/S (in^2/ft)
2.000
1.800
1.600
1.400
1.200
1.000
0.800
0.600
0.400
0.200
0.000
0.00
13.81
27.61
41.42
55.23
69.04
82.84
96.65
110.46
124.26
138.07
Locations (ft)
Legend:
Av/S provided
Av/S required STR I
View Options...

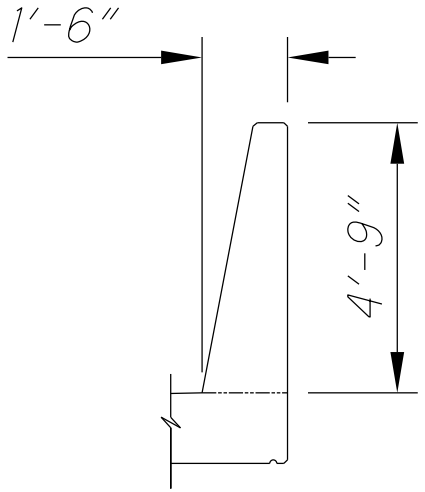


Beam Elevation

BARRIER DIMENSIONS



Left Barrier
Area = 4.08 ft²



Right Barrier
Area = 4.97 ft²

PROJECT DATA

Project:	Innerbelt Bridge No. 6
Designer:	AKS
Date:	Apr/18/2011
Checked By:	DBT
Date Checked:	Apr/18/2011
User job number:	49633
State:	Ohio, State Job #:CUY-90-1627
State Specification:	None
Design Code:	AASHTO LRFD - [5th Edition, with 2010 Interims]
Units:	US
Span Type:	Simple Span
Flared Girder:	No
Comments:	E 14th Street On-Ramp to I-90 WB and Future Mainline I-90 WB over E 9th Street Beam Strength increased to 11 ksi to satisfy rating requirements. Beams cast in shop exceeded that strength.
File Name:	C:\Documents and Settings\asteffl\Desktop\CUY-90-1627-date120210-FINAL.csl

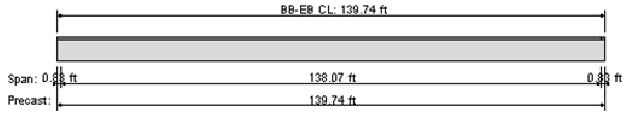
GEOMETRY DATA

BRIDGE LAYOUT

Overall Width (ft)	86.321
Left curb (ft)	1.500
Right curb (ft)	1.500
curb-to-curb width (ft)	83.321
Number of spans	1
Number of lanes	7
Lane width (ft)	12.000
Eff Deck thick (in)	7.500
Sacrificial thick (in)	1.000
Haunch thickness (in)	0.000
Haunch width (in)	48.000
Bridge c/s,MI(Ixx) (in4)	9793146.00

SPAN DATA

Precast length,	ft =	139.740
Bearing-to-bearing,	ft =	138.070
Release span,	ft =	139.740

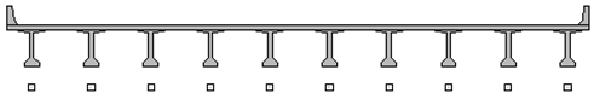


BR01 - Bridge elevation

BEAM DATA

No	ID	Loc-prev ft	Area in2	MI(Ixx) in4	Height in	Yb in	B-topg in	B-trib ft
1	ODOT 66" - 4ft - SHO	3.655	956.0	540204.0	66.00	33.20	48.00	8.071
2	ODOT 66" - 4ft - SHO	8.833	956.0	540204.0	66.00	33.20	48.00	8.833
3	ODOT 66" - 4ft - SHO	8.833	956.0	540204.0	66.00	33.20	48.00	8.833
4	ODOT 66" - 4ft - SHO	8.833	956.0	540204.0	66.00	33.20	48.00	8.833
5	ODOT 66" - 4ft - SHO	8.833	956.0	540204.0	66.00	33.20	48.00	8.833
6	ODOT 66" - 4ft - SHO	8.833	956.0	540204.0	66.00	33.20	48.00	8.833
7	ODOT 66" - 4ft - SHO	8.833	956.0	540204.0	66.00	33.20	48.00	8.833
8	ODOT 66" - 4ft - SHO	8.833	956.0	540204.0	66.00	33.20	48.00	8.833

No	ID	Loc-prev ft	Area in2	MI(Ixx) in4	Height in	Yb in	B-topg in	B-trib ft
9	ODOT 66" - 4ft - SHO	8.833	956.0	540204.0	66.00	33.20	48.00	8.833
10	ODOT 66" - 4ft - SHO	8.833	956.0	540204.0	66.00	33.20	48.00	7.583



BR01 - Bridge cross section

MATERIAL DATA - Project Level

As defined in Material Tab. For beam level properties look at Beam Specific output.

CONCRETE PROPERTIES

	Precast Release	Precast Final	C.I.P
f'c (ksi)	7.500	11.000	4.500
Wc (pcf)	150.000	150.000	150.000
Ec (ksi)	5250.270	6358.390	4066.840
K1	1.000	1.000	1.000

STRAND AND REBAR PROPERTIES

PRESTRESSED STEEL:
6/10-270K-LL, Low relaxation strands
Depressed at 0.45L
Strand Diameter = 0.600 in
Tensile Strength(fpu) = 270.0 ksi
Use transformed strand and rebar: No

REINFORCING STEEL:
Tension/Shear steel: fy = 60.0 ksi Es = 29000 ksi fs = 24.0 ksi

LOADS DATA

Loads generated using Permanent Load Wizard: NO
DEAD LOADS ON PRECAST
UNITS: (Point: kips, Location: ft, Line: klf, Trapez: klf)

Span	Beam	DC/DW	Type	Mag.1	Loc.1	Mag.2	Loc.2	Description
1	1	DC	Line	0.087	0.000	0.087	138.070	Additional Slab
1	1	DC	Line	0.150	0.000	0.150	138.070	Haunch
1	1	DC	Line	0.101	0.000	0.101	138.070	Sacrificial Wearing Surface
1	2	DC	Line	0.150	0.000	0.150	138.070	Haunch
1	2	DC	Line	0.110	0.000	0.110	138.070	Sacrificial Wearing Surface
1	3	DC	Line	0.150	0.000	0.150	138.070	Haunch
1	3	DC	Line	0.110	0.000	0.110	138.070	Sacrificial Wearing Surface
1	4	DC	Line	0.150	0.000	0.150	138.070	Haunch
1	4	DC	Line	0.110	0.000	0.110	138.070	Sacrificial Wearing Surface
1	5	DC	Line	0.150	0.000	0.150	138.070	Haunch
1	5	DC	Line	0.110	0.000	0.110	138.070	Sacrificial Wearing Surface
1	6	DC	Line	0.150	0.000	0.150	138.070	Haunch
1	6	DC	Line	0.110	0.000	0.110	138.070	Sacrificial Wearing Surface
1	7	DC	Line	0.150	0.000	0.150	138.070	Haunch
1	7	DC	Line	0.110	0.000	0.110	138.070	Sacrificial Wearing Surface
1	8	DC	Line	0.150	0.000	0.150	138.070	Haunch
1	8	DC	Line	0.110	0.000	0.110	138.070	Sacrificial Wearing Surface
1	9	DC	Line	0.150	0.000	0.150	138.070	Haunch
1	9	DC	Line	0.110	0.000	0.110	138.070	Sacrificial Wearing Surface
1	10	DC	Line	0.087	0.000	0.087	138.070	Additional Slab
1	10	DC	Line	0.150	0.000	0.150	138.070	Haunch
1	10	DC	Line	0.095	0.000	0.095	138.070	Sacrificial Wearing Surface

DIAPHRAGM LOADS

Span	Beam	Load (kips)	Location (ft)
1	1	0.270	34.517
1	1	0.270	69.035
1	1	0.270	103.552
1	2	0.550	34.517
1	2	0.550	69.035
1	2	0.550	103.552
1	3	0.550	34.517
1	3	0.550	69.035
1	3	0.550	103.552
1	4	0.550	34.517
1	4	0.550	69.035
1	4	0.550	103.552
1	5	0.550	34.517
1	5	0.550	69.035

Span	Beam	Load (kips)	Location (ft)
1	5	0.550	103.552
1	6	0.550	34.517
1	6	0.550	69.035
1	6	0.550	103.552
1	7	0.550	34.517
1	7	0.550	69.035
1	7	0.550	103.552
1	8	0.550	34.517
1	8	0.550	69.035
1	8	0.550	103.552
1	9	0.550	34.517
1	9	0.550	69.035
1	9	0.550	103.552
1	10	0.270	34.517
1	10	0.270	69.035
1	10	0.270	103.552

DEAD LOADS ON COMPOSITE

UNITS: (Point: kips, Location: ft, Line: klf, Trapez: klf, Area: ksf, Width: ft)

Span	DC/DW	Type	Mag.1	Loc.1/Width	Mag.2	Loc.2	Description
1	DC	Line	0.612	0.000	0.612	138.070	Left Barrier Weight
1	DC	Line	0.746	0.000	0.746	138.070	Right Barrier Weight
1	DW	Area	0.060	83.797	-	-	Future Wearing Surface

LIVE LOADS

Live load deflection: included.

ID	Type
Design Lane	Design Lane
Design Tandem	Design Tandem
Design Truck	Design Truck

Pedestrian Load - NONE

LIVE LOADS USED

LIVE LOAD LIBRARY: default.cs3

1 ID: Design Lane

Description:	Design Lane as in AASHTO-LRFD
Type:	Design Lane

Lane Load:	Intensity = 0.64 klf,	Width = 10.00 ft

2 ID: Design Tandem

Description:	Design Tandem as in AASHTO-LRFD
Type:	Design Tandem

First Axle Magnitude = 25.00 k,	Wheel Spacing = 6.00 ft,	Truck Width = 10.00 ft

#	Magnitude, k	Max Spacing, ft	Min Spacing, ft	Increment, ft
1	25.00	4.00	4.00	0.00

3 ID: Design Truck

Description:	Design Truck as in AASHTO-LRFD
Type:	Design Truck

First Axle Magnitude = 8.00 k,	Wheel Spacing = 6.00 ft,	Truck Width = 10.00 ft

#	Magnitude, k	Max Spacing, ft	Min Spacing, ft	Increment, ft
1	32.00	14.00	14.00	0.00
2	32.00	30.00	14.00	2.00

4 ID: Fatigue Truck

Description:	Fatigue Truck as in AASHTO-LRFD
Type:	Fatigue Truck

First Axle Magnitude = 8.00 k,	Wheel Spacing = 6.00 ft,	Truck Width = 10.00 ft

#	Magnitude, k	Max Spacing, ft	Min Spacing, ft	Increment, ft
1	32.00	14.00	14.00	0.00
2	32.00	30.00	30.00	0.00

RATING LOADS - NONE

ANALYSIS DATA

ANALYSIS PARAMETERS DATA

Truck impact:	1.330
Lane impact:	1.000
Strength II impact:	1.330
Fatigue impact:	1.150

DISTRIBUTION FACTORS (Art. 4.6.2.2):

Is Span Post-tensioned:	NO
Include Rigid Cross Section Assumption (Art. 4.6.2.2.2d):	YES
ADTT (Average Daily Truck Traffic) :	4465
Percent of the specified force effect :	0.99

NOTE: Beam specific dead and live load DFs are printed in beam level reports.

LOAD FACTORS: (Table 3.4.1-1 & 3.4.1-2)

	Live	DC(max)	DC(min)	DW(max)	DW(min)
Service I:	1.00	1.00	-	1.00	-
Service III:	0.80	1.00	-	1.00	-
Strength I:	1.75	1.25	0.90	1.50	0.65
Fatigue I:	1.50	-	-	-	-

Ductility Factor:	1.00
Redundancy Factor:	1.00
Importance Factor:	1.00

PROJECT DESIGN PARAMETERS

MULTIPLIERS:

Trans len mult:	Bonded	1.00
	Debonded	1.00
Dev len mult:	Bonded	1.60
	Debonded	2.00

Camber & Deflection Multiplier (PCI ref.)

	Erection	Final
Prestress:	1.80	2.20
Self. Wt:	1.85	2.40
Deck + Haunch:		2.30
Diaphragm:		3.00
DL-Prec.:		3.00
DL-Comp.:		3.00

MOMENT AND SHEAR PROVISIONS:

Ultimate Moment Capacity, Mr-prvd computed:	Strain Compatibility method.
Horizontal Shear, Beam and Slab effects in Vu:	INCLUDED

STRESS LIMITS (Art. 5.9.4):
STRESS LIMITS AT RELEASE BEFORE LOSSES:

		PRECAST	
Strength		7.50	ksi
Elasticity		5250.3	ksi
Max comp		4.50	ksi
Max tens		-0.20	ksi
Max tens,	w/reinf	-0.66	ksi

STRESS LIMITS AT FINAL AFTER LOSSES:

	PRECAST		DECK	
Strength	11.00	ksi	4.50	ksi
Elasticity	6358.39	ksi	4066.84	ksi

STRESS LIMITS AT FINAL 1 (P/S + DL + LL):

	PRECAST		DECK	
Max comp	6.60	ksi	2.70	ksi

STRESS LIMITS AT FINAL 2 (P/S + DL):

	PRECAST		DECK	
Max comp	4.95	ksi	2.02	ksi

FATIGUE I STRESS LIMITS AT FINAL 3 (50% P/S + 50% DL + F_LL):

	PRECAST		DECK	
Max comp	4.40	ksi	-	ksi

SERVICE III (Tension):

	PRECAST		DECK	
Max tens	-0.31	ksi	-0.40	ksi

RESISTANCE FACTORS (Art. 5.5.4.2):

Flexure Reinforced	
Compression controlled sections	0.75
Tension controlled sections	0.90
Flexure Prestressed	
Compression controlled sections	0.75
Tension controlled sections	1.00
Shear	0.90

PRESTRESS LOSSES:

Time Dependent Losses, Approximate Method (Art.5.9.5.3)
Days to release = 0.75
Rel. Humid.(RH) = 75.0 %

RATING PARAMETERS

Rating Factors	References	Values
Condition Factor	LRFR - Table 6A.4.2.3-1	1.00
System Factor for Flexural Effect	LRFR - Table 6A.4.2.4-1	1.00
System Factor for Shear Effect	LRFR - Art. 6A.4.2.4	1.00
ADTT	LRFD - Section C3.6.1.1.2	4465
Dynamic Load Factor for Design Level	LRFR - Art. 6A.4.3.3	0.33
Dynamic Load Factor for Legal and Permit Level	LRFR - Table C6A.4.4.3-1	0.33

For Flexural Effect: Condition Factor * System Factor = 1.00 >= 0.85 LRFR (Art. 6A.4.2.1) OK
For Shear Effect: Condition Factor * System Factor = 1.00 >= 0.85 LRFR (Art. 6A.4.2.1 and 6A.4.2.4) OK

Dead Load Factors (Table 6A.4.2.2-1)

Limit State	DC	DW
Strength I	1.25	1.50
Strength II	1.25	1.50
Service I	1.00	1.00
Service III	1.00	1.00

Allowable Stresses (ksi)

Rating Level	Concrete Compression	Concrete Tension	Steel
Design Inventory	0.60 x f'c = 6.60	0.19 x sqrt(f'c) = 0.63	0.90 x f'y = 218.70
Design Operating	0.60 x f'c = 6.60	0.19 x sqrt(f'c) = 0.63	0.90 x f'y = 218.70
Legal	0.60 x f'c = 6.60	0.19 x sqrt(f'c) = 0.63	-
Permit	0.60 x f'c = 6.60	0.19 x sqrt(f'c) = 0.63	0.90 x f'y = 218.70

Consider shear reinf. across plane (FDOT alternative): No

PROPERTIES

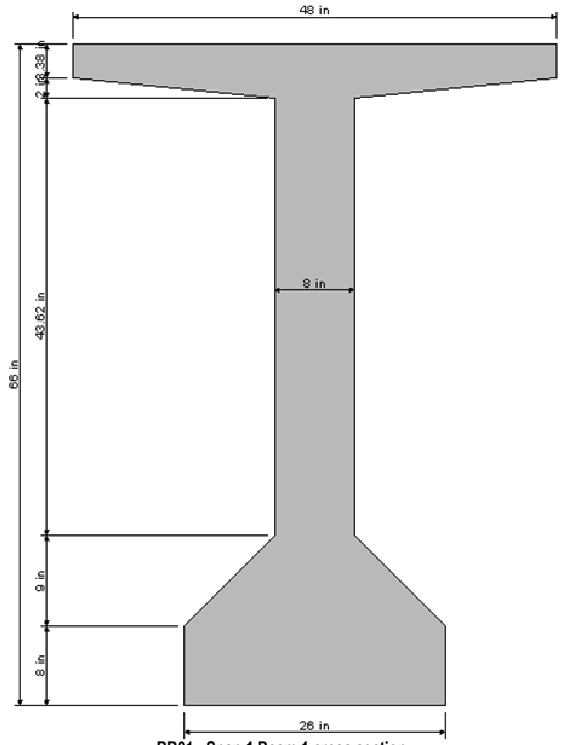
Span:1, Beam:1

PRECAST DATA:

Section Id	ODOT 66" - 4ft - SHOP					
Type	I-Girder					
Flng width	Top	48.000	in	Bot	26.000	in
thick	Top	3.380	in	Bot	8.000	in
Stems	No	1				
	Top	8.000	in			
	Bot	8.000	in			
Shear width		8.000	in			

Article 5.14.1.2.2 checked: OK.

Bentley			Sheet #	DS-12
			Job #	49633
Program:	LEAP® CONSPAN® V8i (SELECTseries 2)	Great Lakes Client Licenses	Designed By	AKS
Version:	10.00.02.19	Copyright © Bentley Systems, Inc. 2011	Date	Apr/18/2011
		www.bentley.com Phone: 1-800-778-4277	Checked	DBT
File Name:	**CUY-90-1627-date120210-FINAL.csl**		Date	Apr/18/2011



BR01 - Span 1 Beam 1 cross section

GENERAL BRIDGE DATA:

Bridge Width	86.32	ft
Curb-to-curb	83.32	ft
Beam Spac. Lt./Rt	3.65/ 8.83	ft
Lane width	12.00	ft
Number of lanes	7	
Interior/Exterior	Exterior	
Start Skew Angle	12.12	degrees
End Skew Angle	12.12	degrees

TOPPING DATA:

Deck	Thickness	7.500	in	
Haunch:				
	Thickness	0.000	in	
	Width	48.000	in	
Effective	width	96.855	in	(Art. 4.6.2.6.1)

GENERAL LOAD DATA:
DEAD LOADS ON PRECAST
UNITS: (Point: kips, Location: ft, Line: klf, Trapez: klf)

DC/DW	Type	Mag.1	Loc.1	Mag.2	Loc.2	Description
DC	Line	0.087	0.000	0.087	138.070	Additional Slab
DC	Line	0.150	0.000	0.150	138.070	Haunch
DC	Line	0.101	0.000	0.101	138.070	Sacrificial Wearing Surface

Diaphragm loads:
(kips, ft)

Mag.	Loc.
0.27	34.52
0.27	69.03
0.27	103.55

Dead loads on composite: See Project info for composite loads

GENERAL SPAN DATA:

Overall length	139.740	ft
Release length	139.740	ft
Design length	138.070	ft

KERN POINTS:

Upper	50.22	in
Lower	15.97	in

DISTRIBUTION FACTORS (Art. 4.6.2.2):
Type k, with deck

Live Moment	(2+ lanes loaded)	0.713	(Calculated)	
Live Moment	(1 lane loaded)	0.805	(Calculated)	(#)
Live Shear	(2+ lanes loaded)	0.745	(Calculated)	
Live Shear	(1 lane loaded)	0.828	(Calculated)	(#)

(#) Lever rule (C4.6.2.2.1)

Pedestrian	0.094	(Calculated)
Comp. DC	0.094	(Calculated)
Comp. DW	0.094	(Calculated)

Dead Loads and Pedestrian Load distributed based on Tributary Fraction
RESISTANCE FACTORS (Art. 5.5.4.2):

Flexure Reinforced	
Compression controlled sections	0.75
Tension controlled sections	0.90
Flexure Prestressed	
Compression controlled sections	0.75
Tension controlled sections	1.00
Shear	0.90

SECTION PROPERTIES:

	PRECAST		COMPOSITE		
Area	956.0	in2	1420.6	in2	#
Total Height	66.00	in	73.50	in	
Mom. of Inertia (Ixx)	540204	in4	960068	in4	#
Ht. of c.g.	33.20	in	45.15	in	#
Density	150.00	pcf	150.00	pcf	
Self-weight	995.8	plf	1752.5	plf	
Mom. of Inertia (Iyy)	58471.3	in4			
Poisson's Ratio	0.2				

(#) Of Total Section using Ect/Ec = 0.6396
Use transformed strand and rebar: No

Span:1, Beam:1
STRESS LIMITS (Art. 5.9.4):
STRESS LIMITS AT RELEASE BEFORE LOSSES:

		PRECAST	
Strength		7.50	ksi
Elasticity		5250.3	ksi
Max comp		4.50	ksi
Max tens		-0.20	ksi

Max tens,	w/reinf	PRECAST -0.66	ksi	

STRESS LIMITS AT FINAL AFTER LOSSES:

	PRECAST		DECK	
Strength	11.00	ksi	4.50	ksi
Elasticity	6358.39	ksi	4066.84	ksi

STRESS LIMITS AT FINAL 1 (P/S + DL + LL):

	PRECAST		DECK	
Max comp	6.60	ksi	2.70	ksi

STRESS LIMITS AT FINAL 2 (P/S + DL):

	PRECAST		DECK	
Max comp	4.95	ksi	2.03	ksi

FATIGUE I STRESS LIMITS AT FINAL 3 (50% P/S + 50% DL + F_LL):

	PRECAST		DECK	
Max comp	4.40	ksi	-	ksi

SERVICE III (Tension):

	PRECAST		DECK	
Max tens	-0.31	ksi	-0.40	ksi

Span:1, Beam:1
PRESTRESSED STEEL:
58 strands, 6/10-270K-LL, Low relaxation strands
Depressed at 0.45L (62.88 ft from member end)

END PATTERN (Ycg = 19.57 in):

11 @ 2.000 in 6 @ 10.000 in 3 @ 59.000 in	8 @ 4.000 in 2 @ 12.000 in 3 @ 61.000 in	8 @ 6.000 in 3 @ 55.000 in 3 @ 63.000 in	8 @ 8.000 in 3 @ 57.000 in

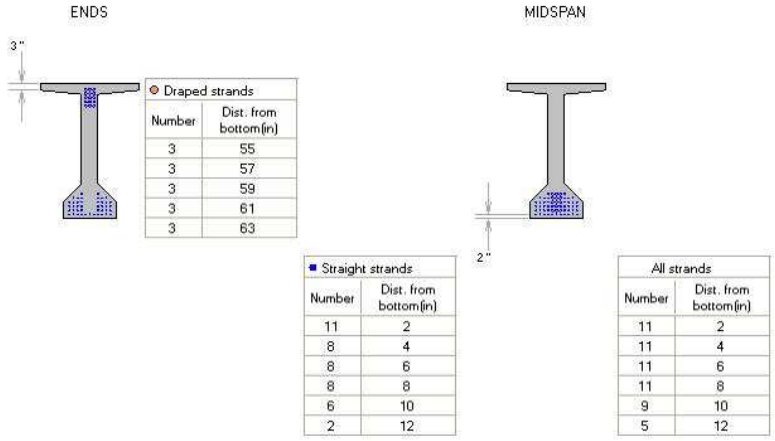
MID PATTERN (Ycg = 6.38 in):
(A) Draped:

3 @ 4.000 in 3 @ 12.000 in	3 @ 6.000 in	3 @ 8.000 in	3 @ 10.000 in

(B) Straight:

11 @ 2.000 in 6 @ 10.000 in	8 @ 4.000 in 2 @ 12.000 in	8 @ 6.000 in	8 @ 8.000 in

Strand Diameter	0.600	in
Strand Area	0.217	in2
Total Strand Area	12.586	in2
Trans. Len,bonded	3.000	ft
Trans. Len,debonded	3.000	ft
Dev. Len, bonded	12.269	ft
Dev. Len, debonded	15.336	ft
Holddown Force	44.447	kips
Tensile Strength(fpu)	270.0	ksi
Initial Prestress = 0.75fpu	202.5	ksi
Initial Pull	2548.7	kips
Beam Shrtng (PL/AE)	0.760	in



BR01 - Strand Pattern, Span 1, Beam 1

Span:1, Beam:1
ESTIMATED QUANTITIES

Prestressing (linear ft)	Strands (LB/1000ft)	(LB)	Beam Vol(C.Y.)	Concrete Wt(LB)	Stirrups (LB)	Longitudinal Bars (LB)
8109.223	740	6000.825	34.360	139158.047	797.368	0.000

Span:1, Beam:1
REINFORCING STEEL:

Tension	steel:	
fy	60.0	ksi
Es	29000	ksi
fs	24.0	ksi

Stirrups:

# legs	Size	fy (ksi)	Area (in2)	Spacing (in)	Start (ft)	End (ft)	Extends into Deck
2	US#4[M13]	60.0	0.40	3.00	0.0000	2.5000	Yes
2	US#4[M13]	60.0	0.40	6.00	2.5000	3.5000	Yes
2	US#4[M13]	60.0	0.40	21.00	3.5000	136.2402	Yes
2	US#4[M13]	60.0	0.40	6.00	136.2402	137.2402	Yes
2	US#4[M13]	60.0	0.40	3.00	137.2402	139.7402	Yes

LOSSES
Note: Values are calculated at Midspan

Str. area	12.5860	in2
Ycg	6.38	in
P_init	2548.7	kips
Ecc	26.82	in
Days to release	0.75	
Rel. Humid.(RH)	75.0	%
Es	28500.0	ksi
Eci	5250	ksi

AASHTO LOSSES

Elastic Shortening 21.69 ksi (Eq 5.9.5.2.3a-1), (fcgp= 3.995 ksi)

	Elastic Gains	Gains		Adjustment	
due	to Precast Loads	-7.02	ksi	0.80	ksi
due	to Composite Loads	-3.09	ksi	0.35	ksi
due	to Live Loads	-6.24	ksi	0.89	ksi

Time Dependent Losses (Approximate Method (Art.5.9.5.3))

		Initial		Final		
Steel relaxation	0.00	ksi		2.40	ksi	(Eq 5.9.5.3-1)
Concrete shrinkage	0.00	ksi		6.71	ksi	(Eq 5.9.5.3-1)

		Initial			Final		
Concrete creep	0.00	ksi			14.90	ksi	(Eq 5.9.5.3-1)
Sub-total	21.69	ksi	(10.71 %)		9.69	ksi	(4.79 %)
Total Prestress Losses					31.38	ksi	(15.50 %)

Prestressing Stress Limit Check (Table 5.9.3.1)
initial fpi = 202.5 ksi < 0.75 fpu, OK
initial fpe = 171.1 ksi < 0.80 fpy, OK

SHEAR/MOMENT ENVELOPE (&REACTIONS)

SHEAR AND MOMENT ENVELOPE : Span : 1, Beam : 1, SERVICE I
Shears: kips, Moments: kft

		Bearing	Trans	H/2	0.10L	0.20L	0.30L	0.40L	Midspan
Location,	ft	0.00	2.17	3.06	13.14	27.11	41.09	55.06	69.04
Self wt. :	M	0.0	146.5	205.9	817.3	1497.9	1984.1	2275.8	2373.0
(Max)	V	68.7	66.6	65.7	55.7	41.7	27.8	13.9	0.0
DL-Prec. :	M	0.0	49.7	69.9	277.3	508.3	673.2	772.2	805.2
DC(Max)	V	23.3	22.6	22.3	18.9	14.2	9.4	4.7	0.0
DL-Prec. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DW(Max)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Deck + :	M	0.0	111.3	156.4	621.0	1138.2	1507.6	1729.2	1803.1
Haunch (Max)	V	52.2	50.6	49.9	42.3	31.7	21.1	10.6	0.0
Diaphragm :	M	0.0	0.9	1.2	5.3	11.0	14.9	16.8	18.6
(Max)	V	0.4	0.4	0.4	0.4	0.4	0.1	0.1	0.1
DL-Comp :	M	0.0	18.7	26.3	104.2	191.0	253.0	290.2	302.6
DC(Max)	V	8.8	8.5	8.4	7.1	5.3	3.5	1.8	0.0
DL-Comp :	M	0.0	69.2	97.2	385.8	707.1	936.6	1074.3	1120.2
DW(Max)	V	32.5	31.4	31.0	26.3	19.7	13.1	6.6	0.0
LL + I :	M+	0.0	228.4	320.8	1269.3	2312.5	3037.7	3466.9	3590.2
	V	110.5	108.1	107.1	95.8	80.5	29.9	8.4	28.2
LL + I :	M-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
LL + I :	Vmx	110.5	108.1	107.1	95.9	81.7	68.2	55.4	43.4
	M	0.0	225.2	315.9	1224.9	2154.6	2724.5	2965.9	2916.1
Total :	M+	0.0	624.6	877.7	3480.3	6366.0	8407.1	9625.3	10012.9
	V	296.5	288.2	284.8	246.5	193.6	105.2	46.1	28.3
Total :	M-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total :	Vmx	296.5	288.2	284.8	246.5	194.8	143.4	93.1	43.6
	M	0.0	621.5	872.7	3436.0	6208.1	8093.9	9124.4	9338.8

		0.60L	0.70L	0.80L	0.90L	H/2	Trans	Bearing
Location,	ft	83.01	96.98	110.96	124.93	135.01	135.91	138.07
Self wt. :	M	2275.8	1984.1	1497.9	817.3	205.9	146.5	0.0
(Max)	V	13.9	27.8	41.7	55.7	65.7	66.6	68.7
DL-Prec. :	M	772.2	673.2	508.3	277.3	69.9	49.7	0.0
DC(Max)	V	4.7	9.4	14.2	18.9	22.3	22.6	23.3
DL-Prec. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DW(Max)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Deck + :	M	1729.2	1507.6	1138.2	621.0	156.4	111.3	0.0
Haunch (Max)	V	10.6	21.1	31.7	42.3	49.9	50.6	52.2
Diaphragm :	M	16.8	14.9	11.0	5.3	1.2	0.9	-0.0
(Max)	V	0.1	0.1	0.4	0.4	0.4	0.4	0.4
DL-Comp :	M	290.2	253.0	191.0	104.2	26.3	18.7	-0.0
DC(Max)	V	1.8	3.5	5.3	7.1	8.4	8.5	8.8
DL-Comp :	M	1074.3	936.6	707.1	385.8	97.2	69.2	0.0
DW(Max)	V	6.6	13.1	19.7	26.3	31.0	31.4	32.5

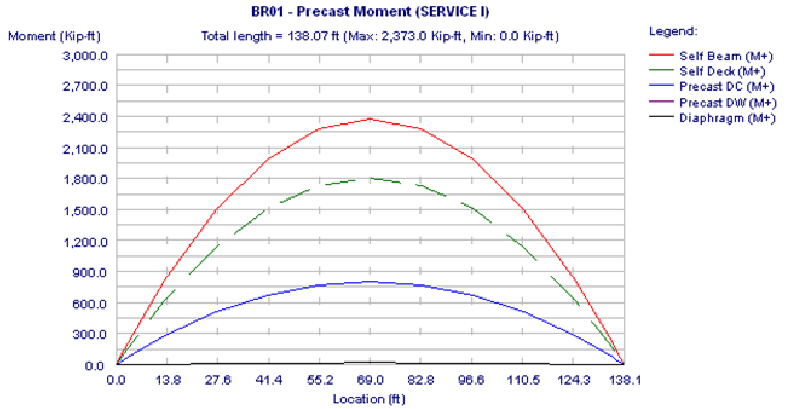
		0.60L	0.70L	0.80L	0.90L	H/2	Trans	Bearing
LL + I :	M+	3466.9	3037.7	2312.5	1269.3	320.8	228.4	-0.0
	V	8.4	29.9	80.5	95.8	107.1	108.1	110.5
LL + I :	M-	0.0	0.0	0.0	0.0	-0.0	-0.0	-0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
LL + I :	Vmx	55.4	68.2	81.7	95.9	107.1	108.1	110.5
	M	2965.9	2724.5	2154.6	1224.9	315.9	225.2	0.0
Total :	M+	9625.3	8407.1	6366.0	3480.3	877.7	624.6	0.0
	V	46.1	105.2	193.6	246.5	284.8	288.2	296.5
Total :	M-	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total :	Vmx	93.1	143.4	194.8	246.5	284.8	288.2	296.5
	M	9124.4	8093.9	6208.1	3436.0	872.7	621.5	0.0

REACTIONS (kips), SERVICE I

Load Type	Left Support	Right Support
Self Wt.	68.7	68.7
Deck+Haunch	52.2	52.2
Diaphragm	0.4	0.4
DL-Prec.(DC)	23.3	23.3
DL-Prec.(DW)	0.0	0.0
DL-Comp.(DC)	93.7	93.7
DL-Comp.(DW)	347.1	347.1
Live	111.3	111.3
Pedestrian	0.0	0.0

Upward reactions are positive.
Live Load reactions are per lane with no distribution factor and no impact.
Non-composite load types are per beam.
Composite and Pedestrian load types are per total bridge width.

Bentley			Sheet #	DS-21
			Job #	49633
Program:	LEAP® CONSPAN® V8i (SELECTseries 2)	Great Lakes Client Licenses	Designed By	AKS
Version:	10.00.02.19	Copyright © Bentley Systems, Inc. 2011	Date	Apr/18/2011
		www.bentley.com Phone: 1-800-778-4277	Checked	DBT
File Name:	**CUY-90-1627-date120210-FINAL.csl**		Date	Apr/18/2011



BR01 - Precast Moment, Span 1, Beam 1, SERVICE I

Bentley			Sheet #	DS-22
			Job #	49633
Program:	LEAP® CONSPAN® V8i (SELECTseries 2)	Great Lakes Client Licenses	Designed By	AKS
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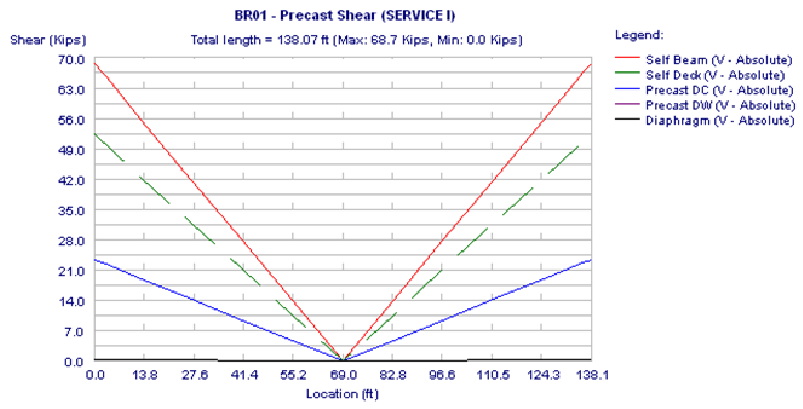
File Name:	**CUY-90-1627-date120210-FINAL.csl**		Date	Apr/18/2011



BR01 - Precast Shear, Span 1, Beam 1, SERVICE I

Bentley			Sheet #	DS-23
			Job #	49633
Program:	LEAP® CONSPAN® V8i (SELECTseries 2)	Great Lakes Client Licenses	Designed By	AKS
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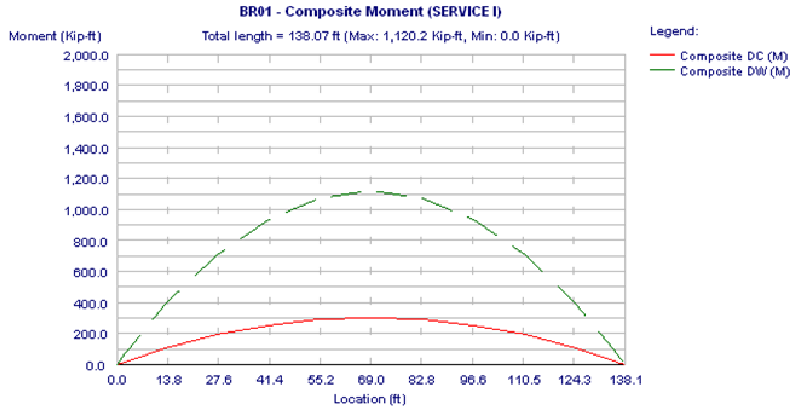
File Name:	**CUY-90-1627-date120210-FINAL.csl**		Date	Apr/18/2011



BR01 - Composite Moment, Span 1, Beam 1, SERVICE I

Bentley			Sheet #	DS-24
			Job #	49633
Program:	LEAP® CONSPAN® V8i (SELECTseries 2)	Great Lakes Client Licenses	Designed By	AKS
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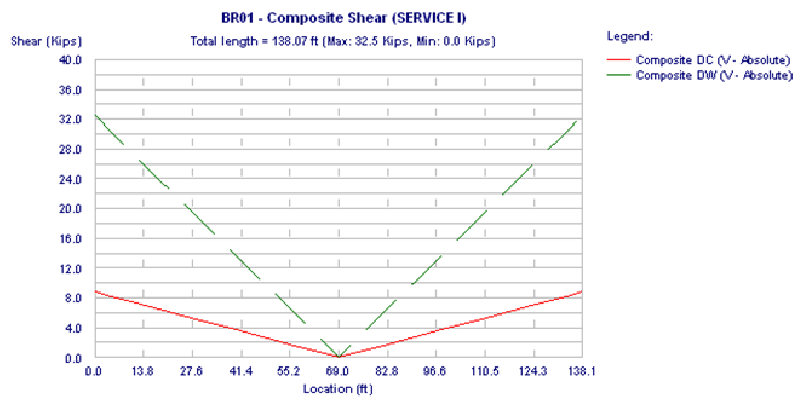
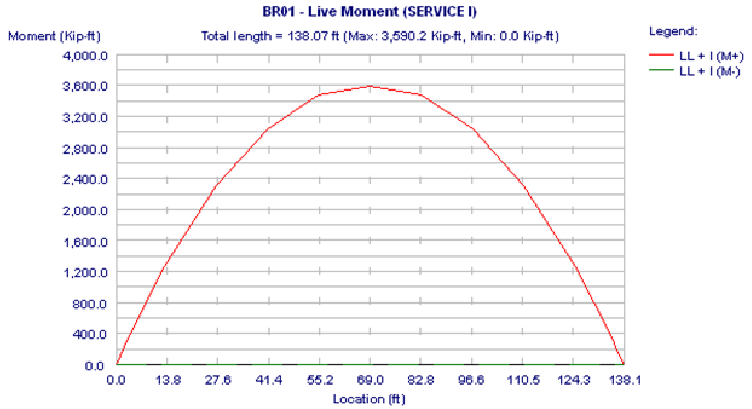
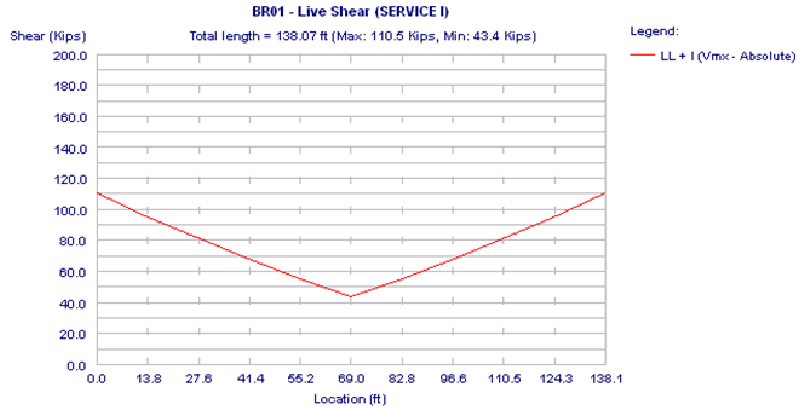
		www.bentley.com Phone: 1-800-778-4277	Checked	DBT
File Name:	**CUY-90-1627-date120210-FINAL.csl**		Date	Apr/18/2011



BR01 - Composite Shear, Span 1, Beam 1, SERVICE I



BR01 - Live Moment, Span 1, Beam 1, SERVICE I



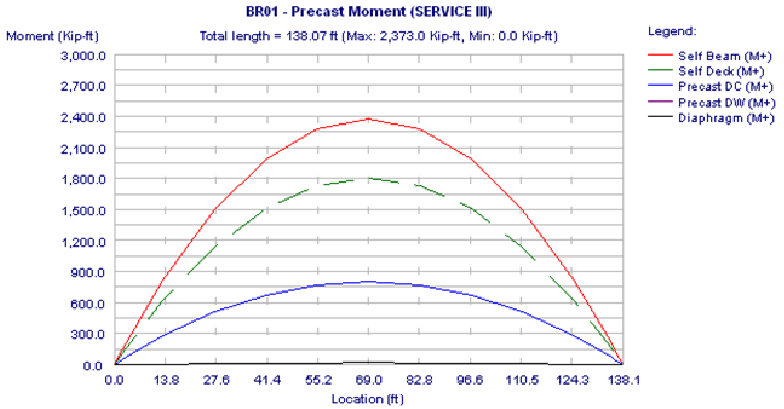
BR01 - Live Shear, Span 1, Beam 1, SERVICE I

SHEAR AND MOMENT ENVELOPE : Span : 1, Beam : 1, SERVICE III
Shears: kips, Moments: kft

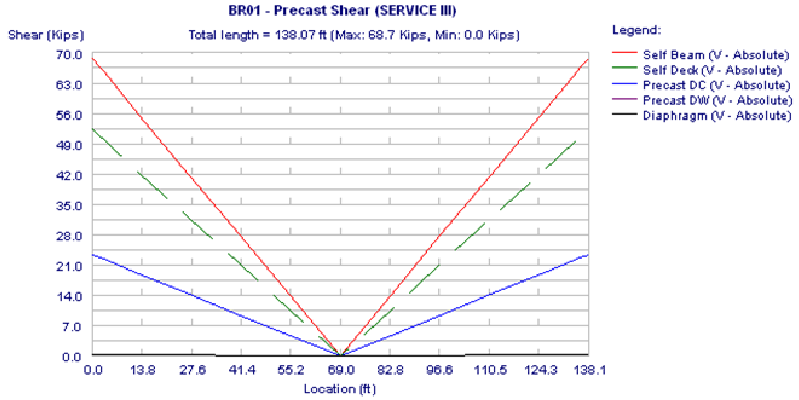
		Bearing	Trans	H/2	0.10L	0.20L	0.30L	0.40L	Midspan
Location,	ft	0.00	2.17	3.06	13.14	27.11	41.09	55.06	69.04
Self wt. :	M	0.0	146.5	205.9	817.3	1497.9	1984.1	2275.8	2373.0
(Max)	V	68.7	66.6	65.7	55.7	41.7	27.8	13.9	0.0
DL-Prec. :	M	0.0	49.7	69.9	277.3	508.3	673.2	772.2	805.2
DC(Max)	V	23.3	22.6	22.3	18.9	14.2	9.4	4.7	0.0
DL-Prec. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DW(Max)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Deck + :	M	0.0	111.3	156.4	621.0	1138.2	1507.6	1729.2	1803.1
Haunch (Max)	V	52.2	50.6	49.9	42.3	31.7	21.1	10.6	0.0
Diaphragm :	M	0.0	0.9	1.2	5.3	11.0	14.9	16.8	18.6
(Max)	V	0.4	0.4	0.4	0.4	0.4	0.1	0.1	0.1
DL-Comp :	M	0.0	18.7	26.3	104.2	191.0	253.0	290.2	302.6
DC(Max)	V	8.8	8.5	8.4	7.1	5.3	3.5	1.8	0.0
DL-Comp :	M	0.0	69.2	97.2	385.8	707.1	936.6	1074.3	1120.2
DW(Max)	V	32.5	31.4	31.0	26.3	19.7	13.1	6.6	0.0

		Bearing	Trans	H/2	0.10L	0.20L	0.30L	0.40L	Midspan
LL + I :	M+	0.0	182.7	256.7	1015.4	1850.0	2430.2	2773.5	2872.1
	V	88.4	86.5	85.7	76.7	64.4	23.9	6.7	22.5
LL + I :	M-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
LL + I :	Vmx	88.4	86.5	85.7	76.7	65.4	54.5	44.3	34.7
	M	0.0	180.2	252.7	979.9	1723.7	2179.6	2372.8	2332.9
Total :	M+	0.0	579.0	813.5	3226.5	5903.5	7799.5	8931.9	9294.8
	V	274.3	266.6	263.4	227.3	177.5	99.2	44.4	22.7
Total :	M-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total :	Vmx	274.3	266.6	263.4	227.3	178.4	129.8	82.0	34.9
	M	0.0	576.4	809.5	3191.0	5777.2	7549.0	8531.2	8755.6

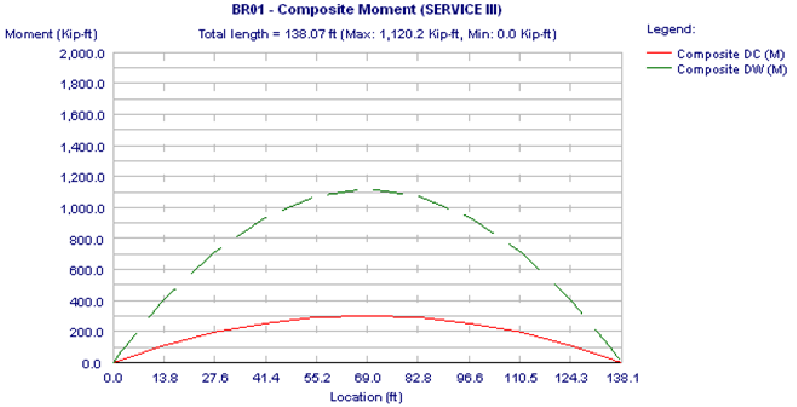
		0.60L	0.70L	0.80L	0.90L	H/2	Trans	Bearing
Location,	ft	83.01	96.98	110.96	124.93	135.01	135.91	138.07
Self wt. :	M	2275.8	1984.1	1497.9	817.3	205.9	146.5	0.0
(Max)	V	13.9	27.8	41.7	55.7	65.7	66.6	68.7
DL-Prec. :	M	772.2	673.2	508.3	277.3	69.9	49.7	0.0
DC(Max)	V	4.7	9.4	14.2	18.9	22.3	22.6	23.3
DL-Prec. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DW(Max)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Deck + :	M	1729.2	1507.6	1138.2	621.0	156.4	111.3	0.0
Haunch (Max)	V	10.6	21.1	31.7	42.3	49.9	50.6	52.2
Diaphragm :	M	16.8	14.9	11.0	5.3	1.2	0.9	-0.0
(Max)	V	0.1	0.1	0.4	0.4	0.4	0.4	0.4
DL-Comp :	M	290.2	253.0	191.0	104.2	26.3	18.7	-0.0
DC(Max)	V	1.8	3.5	5.3	7.1	8.4	8.5	8.8
DL-Comp :	M	1074.3	936.6	707.1	385.8	97.2	69.2	0.0
DW(Max)	V	6.6	13.1	19.7	26.3	31.0	31.4	32.5
LL + I :	M+	2773.5	2430.2	1850.0	1015.4	256.7	182.7	-0.0
	V	6.7	23.9	64.4	76.7	85.7	86.5	88.4
LL + I :	M-	0.0	0.0	0.0	0.0	-0.0	-0.0	-0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
LL + I :	Vmx	44.3	54.5	65.4	76.7	85.7	86.5	88.4
	M	2372.8	2179.6	1723.7	979.9	252.7	180.2	0.0
Total :	M+	8931.9	7799.5	5903.5	3226.5	813.5	579.0	0.0
	V	44.4	99.2	177.5	227.3	263.4	266.6	274.3
Total :	M-	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total :	Vmx	82.0	129.8	178.4	227.3	263.4	266.6	274.3
	M	8531.2	7549.0	5777.2	3191.0	809.5	576.4	0.0



BR01 - Precast Moment, Span 1, Beam 1, SERVICE III

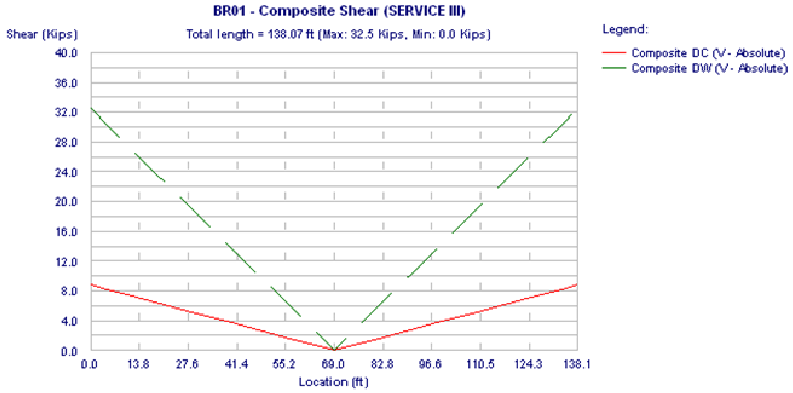


BR01 - Precast Shear, Span 1, Beam 1, SERVICE III



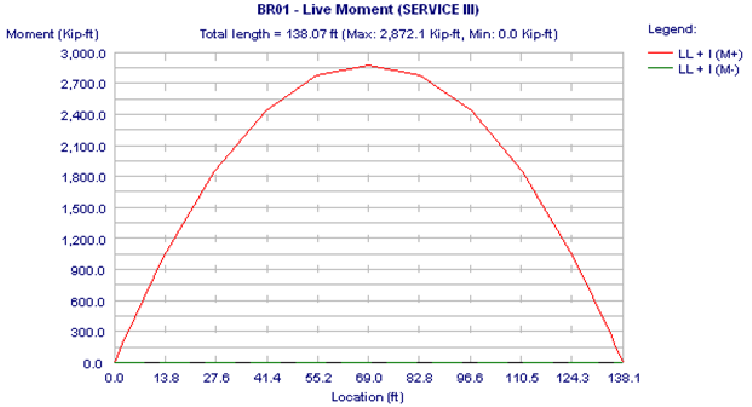
BR01 - Composite Moment, Span 1, Beam 1, SERVICE III

Bentley			Sheet #	DS-31
			Job #	49633
Program:	LEAP® CONSPAN® V8i (SELECTseries 2)	Great Lakes Client Licenses	Designed By	AKS
Version:	10.00.02.19	Copyright © Bentley Systems, Inc. 2011	Date	Apr/18/2011
		www.bentley.com Phone: 1-800-778-4277	Checked	DBT
File Name:	**CUY-90-1627-date120210-FINAL.csl**		Date	Apr/18/2011

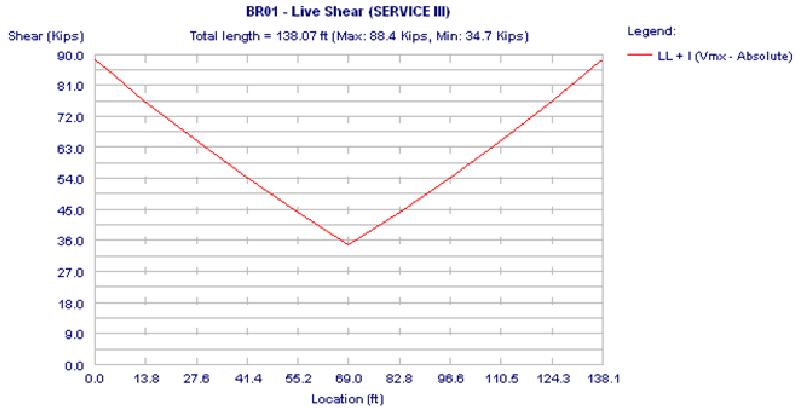


BR01 - Composite Shear, Span 1, Beam 1, SERVICE III

Bentley			Sheet #	DS-32
			Job #	49633
Program:	LEAP® CONSPAN® V8i (SELECTseries 2)	Great Lakes Client Licenses	Designed By	AKS
Version:	10.00.02.19	Copyright © Bentley Systems, Inc. 2011	Date	Apr/18/2011
		www.bentley.com Phone: 1-800-778-4277	Checked	DBT
File Name:	**CUY-90-1627-date120210-FINAL.csl**		Date	Apr/18/2011



BR01 - Live Moment, Span 1, Beam 1, SERVICE III



BR01 - Live Shear, Span 1, Beam 1, SERVICE III

SHEAR AND MOMENT ENVELOPE : Span : 1, Beam : 1, STRENGTH I
Shears: kips, Moments: kft

		Bearing	Trans	H/2	0.10L	0.20L	0.30L	0.40L	Midspan
Location,	ft	0.00	2.17	3.06	13.14	27.11	41.09	55.06	69.04
Self wt. :	M	0.0	183.1	257.3	1021.6	1872.4	2480.1	2844.7	2966.2
(Max)	V	85.9	83.2	82.1	69.6	52.2	34.8	17.4	0.0
Self wt. :	M	0.0	131.9	185.3	735.6	1348.1	1785.7	2048.2	2135.7
(Min)	V	61.9	59.9	59.1	50.1	37.6	25.0	12.5	0.0
DL-Prec. :	M	0.0	62.1	87.3	346.6	635.3	841.5	965.2	1006.5
DC(Max)	V	29.2	28.2	27.9	23.6	17.7	11.8	5.9	0.0
DL-Prec. :	M	0.0	44.7	62.9	249.6	457.4	605.9	695.0	724.6
DC(Min)	V	21.0	20.3	20.1	17.0	12.7	8.5	4.2	0.0
DL-Prec. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DW(Max)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DL-Prec. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DW(Min)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Deck + :	M	0.0	139.2	195.5	776.3	1422.7	1884.5	2161.5	2253.9
Haunch (Max)	V	65.3	63.2	62.4	52.9	39.7	26.4	13.2	0.0

		Bearing	Trans	H/2	0.10L	0.20L	0.30L	0.40L	Midspan
Deck + :	M	0.0	100.2	140.8	558.9	1024.4	1356.8	1556.3	1622.8
Haunch (Min)	V	47.0	45.5	44.9	38.1	28.5	19.0	9.5	0.0
Diaphragm :	M	0.0	1.1	1.6	6.7	13.7	18.6	20.9	23.3
(Max)	V	0.5	0.5	0.5	0.5	0.5	0.2	0.2	0.2
Diaphragm :	M	0.0	0.8	1.1	4.8	9.9	13.4	15.1	16.8
(Min)	V	0.4	0.4	0.4	0.4	0.4	0.1	0.1	0.1
DL-Comp :	M	0.0	23.4	32.8	130.3	238.7	316.2	362.7	378.2
DC(Max)	V	11.0	10.6	10.5	8.9	6.7	4.4	2.2	0.0
DL-Comp :	M	0.0	16.8	23.6	93.8	171.9	227.7	261.2	272.3
DC(Min)	V	7.9	7.6	7.5	6.4	4.8	3.2	1.6	0.0
DL-Comp :	M	0.0	103.7	145.8	578.8	1060.7	1405.0	1611.5	1680.4
DW(Max)	V	48.7	47.2	46.5	39.4	29.6	19.7	9.9	0.0
DL-Comp :	M	0.0	45.0	63.2	250.8	459.6	608.8	698.3	728.2
DW(Min)	V	21.1	20.4	20.2	17.1	12.8	8.5	4.3	0.0
LL + I :	M+	0.0	399.7	561.5	2221.2	4046.8	5316.0	6067.0	6282.8
	V	193.4	189.2	187.4	167.7	140.9	52.4	14.6	49.3
LL + I :	M-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
LL + I :	Vmx	193.4	189.2	187.4	167.7	143.0	119.3	96.9	76.0
	M	0.0	394.2	552.8	2143.6	3770.6	4768.0	5190.4	5103.2
Total :	M+	0.0	912.3	1281.8	5081.5	9290.5	12261.9	14033.7	14591.2
	V	433.9	422.2	417.3	362.6	287.2	149.7	63.4	49.4
Total :	M-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total :	Vmx	433.9	422.2	417.3	362.6	289.3	216.7	145.7	76.2
	M	0.0	906.8	1273.2	5003.9	9014.2	11713.8	13157.0	13411.6

		0.60L	0.70L	0.80L	0.90L	H/2	Trans	Bearing
Location,	ft	83.01	96.98	110.96	124.93	135.01	135.91	138.07
Self wt. :	M	2844.7	2480.1	1872.4	1021.6	257.3	183.1	0.0
(Max)	V	17.4	34.8	52.2	69.6	82.1	83.2	85.9
Self wt. :	M	2048.2	1785.7	1348.1	735.6	185.3	131.9	0.0
(Min)	V	12.5	25.0	37.6	50.1	59.1	59.9	61.9
DL-Prec. :	M	965.2	841.5	635.3	346.6	87.3	62.1	0.0
DC(Max)	V	5.9	11.8	17.7	23.6	27.9	28.2	29.2
DL-Prec. :	M	695.0	605.9	457.4	249.6	62.9	44.7	0.0
DC(Min)	V	4.2	8.5	12.7	17.0	20.1	20.3	21.0
DL-Prec. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DW(Max)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DL-Prec. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DW(Min)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Deck + :	M	2161.5	1884.5	1422.7	776.3	195.5	139.2	0.0
Haunch (Max)	V	13.2	26.4	39.7	52.9	62.4	63.2	65.3
Deck + :	M	1556.3	1356.8	1024.4	558.9	140.8	100.2	0.0
Haunch (Min)	V	9.5	19.0	28.5	38.1	44.9	45.5	47.0
Diaphragm :	M	20.9	18.6	13.7	6.7	1.6	1.1	-0.0
(Max)	V	0.2	0.2	0.5	0.5	0.5	0.5	0.5
Diaphragm :	M	15.1	13.4	9.9	4.8	1.1	0.8	-0.0
(Min)	V	0.1	0.1	0.4	0.4	0.4	0.4	0.4
DL-Comp :	M	362.7	316.2	238.7	130.3	32.8	23.4	-0.0

		0.60L	0.70L	0.80L	0.90L	H/2	Trans	Bearing
DC(Max)	V	2.2	4.4	6.7	8.9	10.5	10.6	11.0
DL-Comp :	M	261.2	227.7	171.9	93.8	23.6	16.8	-0.0
DC(Min)	V	1.6	3.2	4.8	6.4	7.5	7.6	7.9
DL-Comp :	M	1611.5	1405.0	1060.7	578.8	145.8	103.7	0.0
DW(Max)	V	9.9	19.7	29.6	39.4	46.5	47.2	48.7
DL-Comp :	M	698.3	608.8	459.6	250.8	63.2	45.0	0.0
DW(Min)	V	4.3	8.5	12.8	17.1	20.2	20.4	21.1
LL + I :	M+	6067.0	5316.0	4046.8	2221.2	561.5	399.7	-0.0
	V	14.6	52.4	140.9	167.7	187.4	189.2	193.4
LL + I :	M-	0.0	0.0	0.0	0.0	-0.0	-0.0	-0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
LL + I :	Vmx	96.9	119.3	143.0	167.7	187.4	189.2	193.4
	M	5190.4	4768.0	3770.6	2143.6	552.8	394.2	0.0
Total :	M+	14033.7	12261.9	9290.5	5081.5	1281.8	912.3	0.0
	V	63.4	149.7	287.2	362.6	417.3	422.2	433.9
Total :	M-	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total :	Vmx	145.7	216.7	289.3	362.6	417.3	422.2	433.9
	M	13157.0	11713.8	9014.2	5003.9	1273.2	906.8	0.0

REACTIONS (kips), STRENGTH I

Load Type	Left Support	Right Support
Self Wt.	85.9	85.9
Deck+Haunch	65.3	65.3
Diaphragm	0.5	0.5
DL-Prec.(DC)	29.2	29.2
DL-Prec.(DW)	0.0	0.0
DL-Comp.(DC)	117.2	117.2
DL-Comp.(DW)	520.6	520.6
Live	194.8	194.8
Pedestrian	0.0	0.0

Upward reactions are positive.
Live Load reactions are per lane with no distribution factor and no impact.
Non-composite load types are per beam.
Composite and Pedestrian load types are per total bridge width.

Bentley			Sheet #	DS-36
			Job #	49633
Program:	LEAP® CONSPAN® V8i (SELECTseries 2)	Great Lakes Client Licenses	Designed By	AKS
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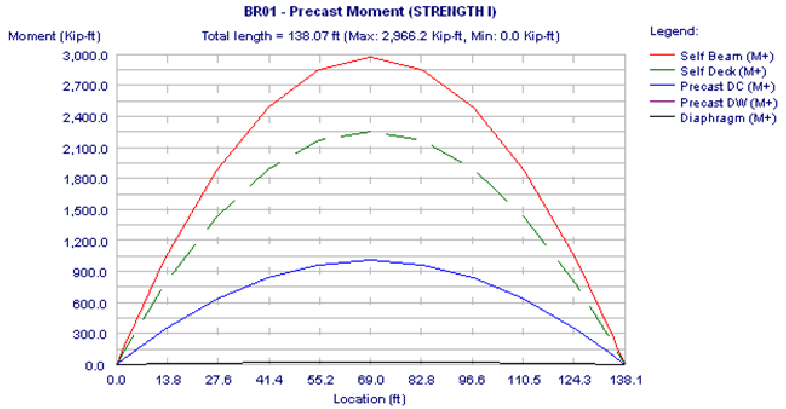
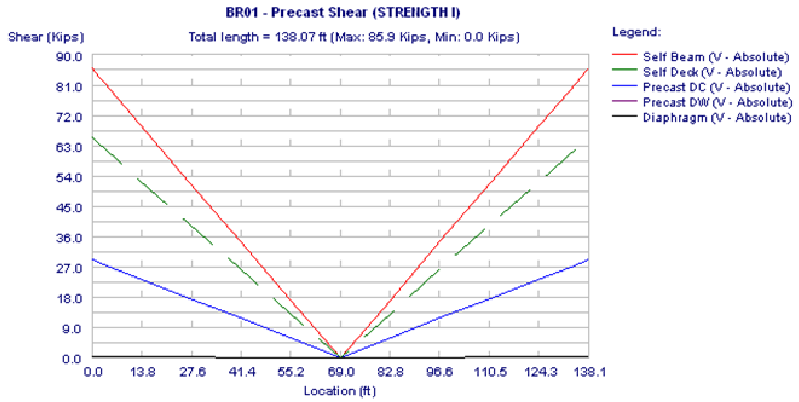
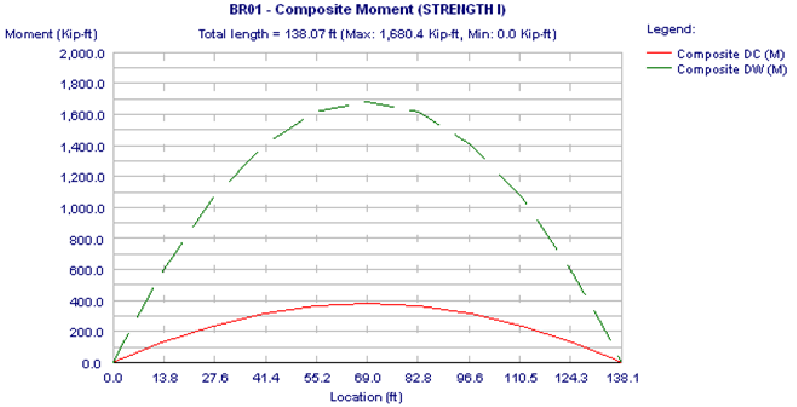
		www.bentley.com Phone: 1-800-778-4277	Checked	DBT
File Name:	CUY-90-1627-date120210-FINAL.csl		Date	Apr/18/2011



BR01 - Precast Moment, Span 1, Beam 1, STRENGTH I

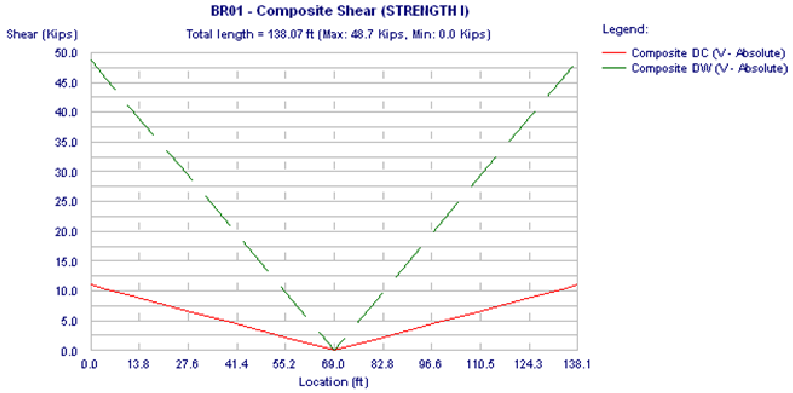


BR01 - Precast Shear, Span 1, Beam 1, STRENGTH I



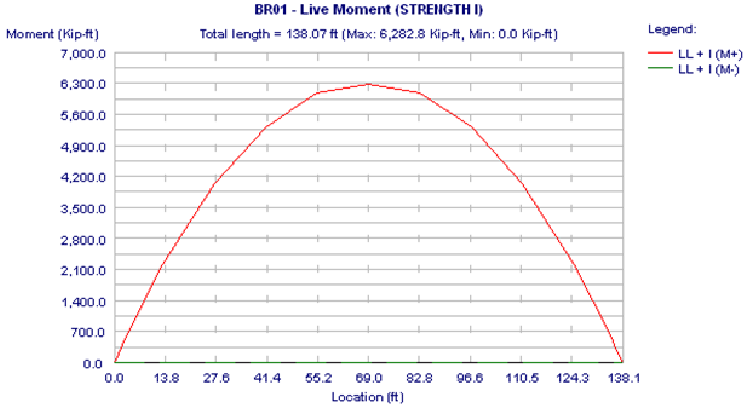
BR01 - Composite Moment, Span 1, Beam 1, STRENGTH I

Bentley			Sheet #	DS-39
			Job #	49633
Program:	LEAP® CONSPAN® V8i (SELECTseries 2)	Great Lakes Client Licenses	Designed By	AKS
Version:	10.00.02.19	Copyright © Bentley Systems, Inc. 2011	Date	Apr/18/2011
		www.bentley.com Phone: 1-800-778-4277	Checked	DBT
File Name:	**CUY-90-1627-date120210-FINAL.csl**		Date	Apr/18/2011

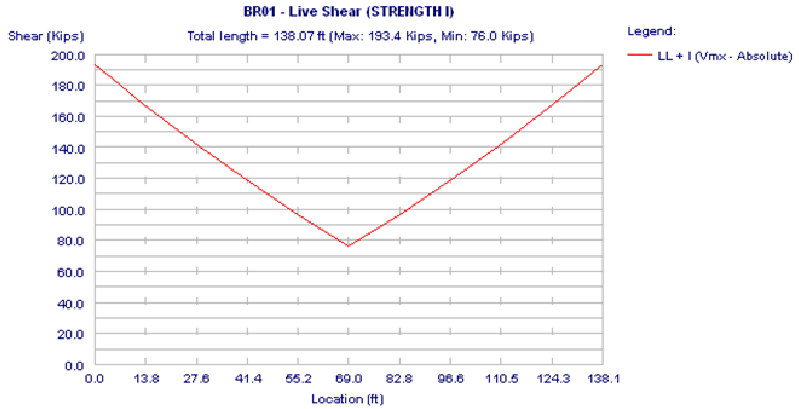


BR01 - Composite Shear, Span 1, Beam 1, STRENGTH I

Bentley			Sheet #	DS-40
			Job #	49633
Program:	LEAP® CONSPAN® V8i (SELECTseries 2)	Great Lakes Client Licenses	Designed By	AKS
Version:	10.00.02.19	Copyright © Bentley Systems, Inc. 2011	Date	Apr/18/2011
		www.bentley.com Phone: 1-800-778-4277	Checked	DBT
File Name:	**CUY-90-1627-date120210-FINAL.csl**		Date	Apr/18/2011



BR01 - Live Moment, Span 1, Beam 1, STRENGTH I



BR01 - Live Shear, Span 1, Beam 1, STRENGTH I

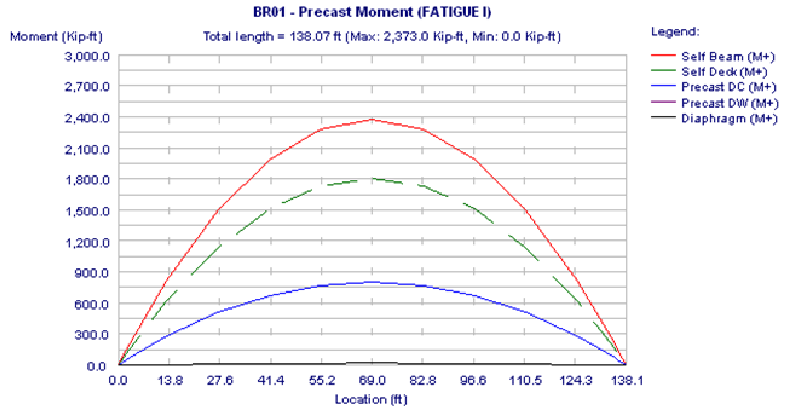
SHEAR AND MOMENT ENVELOPE : Span : 1, Beam : 1, FATIGUE I
Shears: kips, Moments: kft

		Bearing	Trans	H/2	0.10L	0.20L	0.30L	0.40L	Midspan
Location,	ft	0.00	2.17	3.06	13.14	27.11	41.09	55.06	69.04
Self wt. :	M	0.0	146.5	205.9	817.3	1497.9	1984.1	2275.8	2373.0
(Max)	V	68.7	66.6	65.7	55.7	41.7	27.8	13.9	0.0
Self wt. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
(Min)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DL-Prec. :	M	0.0	49.7	69.9	277.3	508.3	673.2	772.2	805.2
DC(Max)	V	23.3	22.6	22.3	18.9	14.2	9.4	4.7	0.0
DL-Prec. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DC(Min)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DL-Prec. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DW(Max)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DL-Prec. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DW(Min)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Deck + :	M	0.0	111.3	156.4	621.0	1138.2	1507.6	1729.2	1803.1
Haunch (Max)	V	52.2	50.6	49.9	42.3	31.7	21.1	10.6	0.0

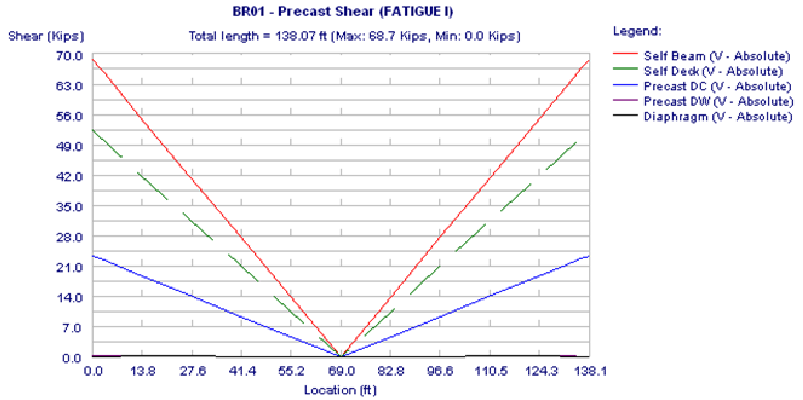
		Bearing	Trans	H/2	0.10L	0.20L	0.30L	0.40L	Midspan
Deck + :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Haunch (Min)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Diaphragm :	M	0.0	0.9	1.2	5.3	11.0	14.9	16.8	18.6
(Max)	V	0.4	0.4	0.4	0.4	0.4	0.1	0.1	0.1
Diaphragm :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
(Min)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DL-Comp :	M	0.0	18.7	26.3	104.2	191.0	253.0	290.2	302.6
DC(Max)	V	8.8	8.5	8.4	7.1	5.3	3.5	1.8	0.0
DL-Comp :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DC(Min)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DL-Comp :	M	0.0	69.2	97.2	385.8	707.1	936.6	1074.3	1120.2
DW(Max)	V	32.5	31.4	31.0	26.3	19.7	13.1	6.6	0.0
DL-Comp :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DW(Min)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
LL + I :	M+	0.0	152.8	214.5	841.1	1509.7	1978.5	2234.0	2256.6
	V	74.4	73.0	72.4	65.8	57.2	43.2	34.6	26.0
LL + I :	M-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
LL + I :	Vmx	74.4	73.0	72.4	65.8	57.2	48.7	40.1	31.5
	M	0.0	152.8	214.5	841.1	1509.7	1945.3	2147.9	2117.6
Total :	M+	0.0	549.0	771.3	3052.1	5563.1	7347.9	8392.5	8679.3
	V	260.3	253.1	250.1	216.4	170.3	118.4	72.3	26.2
Total :	M-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total :	Vmx	260.3	253.1	250.1	216.4	170.3	123.9	77.8	31.7
	M	0.0	549.0	771.3	3052.1	5563.1	7314.6	8306.4	8540.4

		0.60L	0.70L	0.80L	0.90L	H/2	Trans	Bearing
Location,	ft	83.01	96.98	110.96	124.93	135.01	135.91	138.07
Self wt. :	M	2275.8	1984.1	1497.9	817.3	205.9	146.5	0.0
(Max)	V	13.9	27.8	41.7	55.7	65.7	66.6	68.7
Self wt. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0
(Min)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DL-Prec. :	M	772.2	673.2	508.3	277.3	69.9	49.7	0.0
DC(Max)	V	4.7	9.4	14.2	18.9	22.3	22.6	23.3
DL-Prec. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DC(Min)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DL-Prec. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DW(Max)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DL-Prec. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DW(Min)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Deck + :	M	1729.2	1507.6	1138.2	621.0	156.4	111.3	0.0
Haunch (Max)	V	10.6	21.1	31.7	42.3	49.9	50.6	52.2
Deck + :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Haunch (Min)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Diaphragm :	M	16.8	14.9	11.0	5.3	1.2	0.9	-0.0
(Max)	V	0.1	0.1	0.4	0.4	0.4	0.4	0.4
Diaphragm :	M	0.0	0.0	0.0	0.0	0.0	0.0	-0.0
(Min)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DL-Comp :	M	290.2	253.0	191.0	104.2	26.3	18.7	-0.0

		0.60L	0.70L	0.80L	0.90L	H/2	Trans	Bearing
DC(Max)	V	1.8	3.5	5.3	7.1	8.4	8.5	8.8
DL-Comp :	M	0.0	0.0	0.0	0.0	0.0	0.0	-0.0
DC(Min)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DL-Comp :	M	1074.3	936.6	707.1	385.8	97.2	69.2	0.0
DW(Max)	V	6.6	13.1	19.7	26.3	31.0	31.4	32.5
DL-Comp :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DW(Min)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
LL + I :	M+	2234.0	1978.5	1509.7	841.1	214.5	152.8	0.0
	V	34.6	43.2	57.2	65.8	72.4	73.0	74.4
LL + I :	M-	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
LL + I :	Vmx	40.1	48.7	57.2	65.8	72.4	73.0	74.4
	M	2147.9	1945.3	1509.7	841.1	214.5	152.8	0.0
Total :	M+	8392.5	7347.9	5563.1	3052.1	771.3	549.0	0.0
	V	72.3	118.4	170.3	216.4	250.1	253.1	260.3
Total :	M-	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total :	Vmx	77.8	123.9	170.3	216.4	250.1	253.1	260.3
	M	8306.4	7314.6	5563.1	3052.1	771.3	549.0	0.0

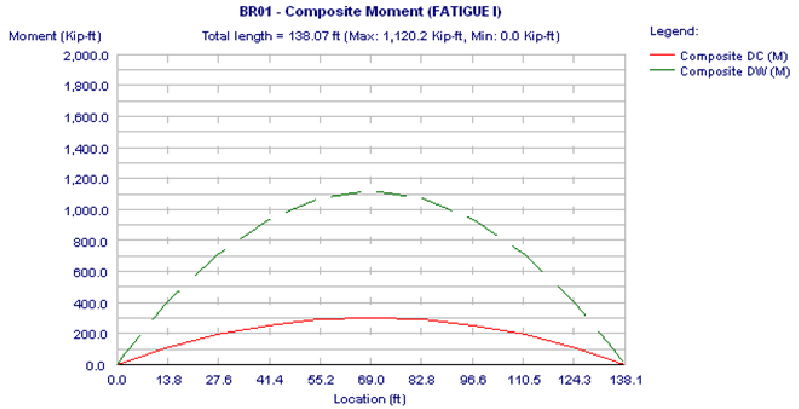


BR01 - Precast Moment, Span 1, Beam 1, FATIGUE I



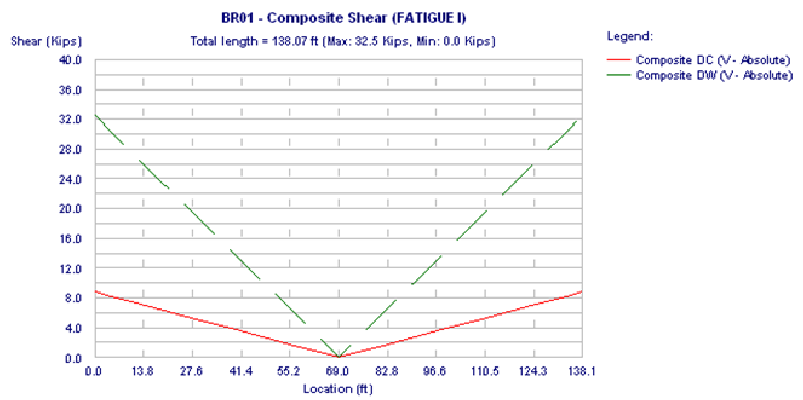
BR01 - Precast Shear, Span 1, Beam 1, FATIGUE I

Bentley			Sheet #	DS-45
			Job #	49633
Program:	LEAP® CONSPAN® V8i (SELECTseries 2)	Great Lakes Client Licenses	Designed By	AKS
Version:	10.00.02.19	Copyright © Bentley Systems, Inc. 2011	Date	Apr/18/2011
		www.bentley.com Phone: 1-800-778-4277	Checked	DBT
File Name:	**CUY-90-1627-date120210-FINAL.csl**		Date	Apr/18/2011



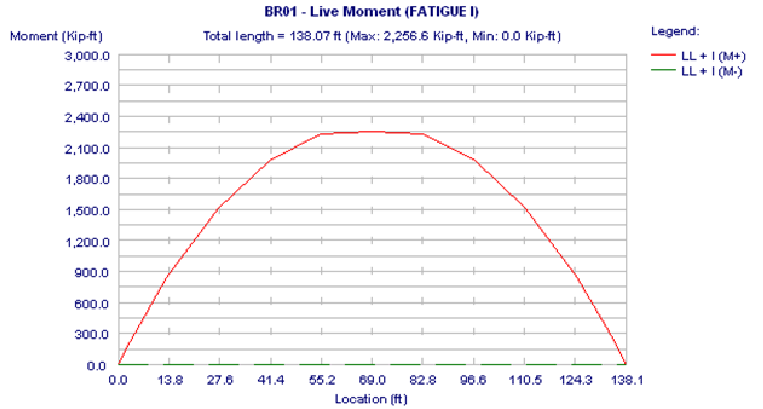
BR01 - Composite Moment, Span 1, Beam 1, FATIGUE I

Bentley			Sheet #	DS-46
			Job #	49633
Program:	LEAP® CONSPAN® V8i (SELECTseries 2)	Great Lakes Client Licenses	Designed By	AKS
Version:	10.00.02.19	Copyright © Bentley Systems, Inc. 2011	Date	Apr/18/2011
		www.bentley.com Phone: 1-800-778-4277	Checked	DBT
File Name:	**CUY-90-1627-date120210-FINAL.csl**		Date	Apr/18/2011



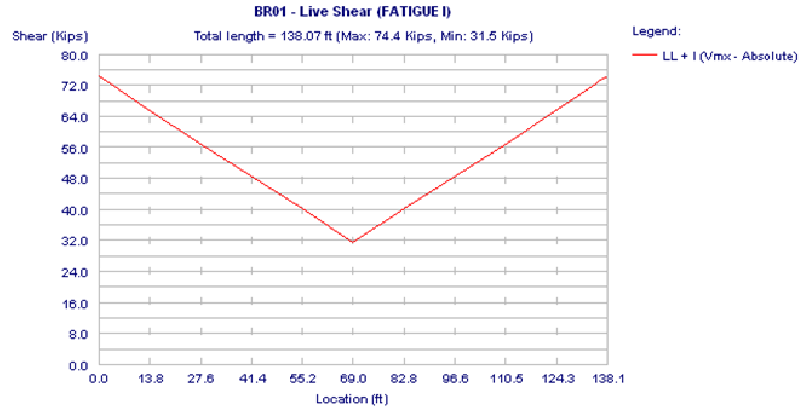
BR01 - Composite Shear, Span 1, Beam 1, FATIGUE I

Bentley			Sheet #	DS-47
			Job #	49633
Program:	LEAP® CONSPAN® V8i (SELECTseries 2)	Great Lakes Client Licenses	Designed By	AKS
Version:	10.00.02.19	Copyright © Bentley Systems, Inc. 2011	Date	Apr/18/2011
		www.bentley.com Phone: 1-800-778-4277	Checked	DBT
File Name:	**CUY-90-1627-date120210-FINAL.csl**		Date	Apr/18/2011



BR01 - Live Moment, Span 1, Beam 1, FATIGUE I

Bentley			Sheet #	DS-48
			Job #	49633
Program:	LEAP® CONSPAN® V8i (SELECTseries 2)	Great Lakes Client Licenses	Designed By	AKS
Version:	10.00.02.19	Copyright © Bentley Systems, Inc. 2011	Date	Apr/18/2011
		www.bentley.com Phone: 1-800-778-4277	Checked	DBT
File Name:	**CUY-90-1627-date120210-FINAL.csl**		Date	Apr/18/2011



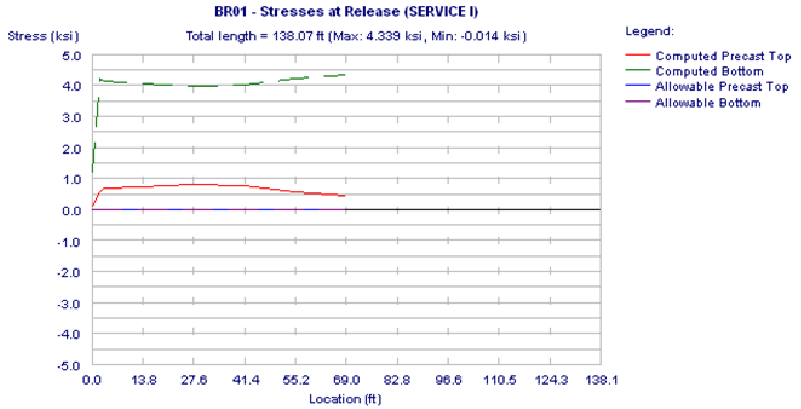
BR01 - Live Shear, Span 1, Beam 1, FATIGUE I

POSITIVE ENVELOPE STRESSES

Span : 1, Beam : 1, SERVICE I

RELEASE STRESSES, (ksi) (LOSS = 10.71 %)

	Trans	0.10L /0.90L	0.20L /0.80L	0.30L /0.70L	0.40L /0.60L	Midspan	Depress.
Location, ft	3.00	13.97	27.95	41.92	55.90	69.87	62.88
Beam-Self							
Precast-top	0.149	0.638	1.133	1.488	1.700	1.771	1.753
Bottom	-0.151	-0.645	-1.147	-1.506	-1.721	-1.793	-1.775
Prestress							
Precast-top	0.410	0.092	-0.313	-0.718	-1.123	-1.326	-1.326
Bottom	4.375	4.697	5.107	5.517	5.927	6.132	6.132
Total							
Precast-top	0.559	0.730	0.820	0.770	0.577	0.446	0.428
Bottom	4.224	4.051	3.959	4.011	4.206	4.339	4.357



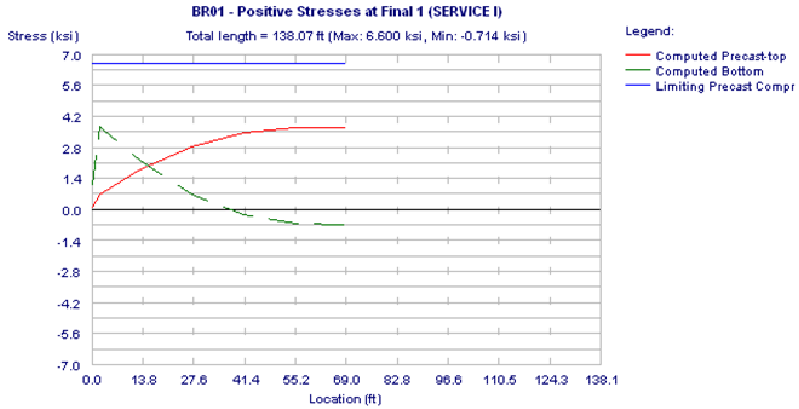
BR01 - Stresses at Release, Span 1, Beam 1, SERVICE I

SERVICE I

POSITIVE ENVELOPE STRESSES, (ksi) (LOSS = 15.50 %)

	Bearing	Trans	H/2	0.10L /0.90L	0.20L /0.80L	0.30L /0.70L	0.40L /0.60L	Midspan
Location, ft	0.00	2.17	3.06	13.14	27.11	41.09	55.06	69.04
Prestress								
Precast-top	0.125	0.388	0.363	0.087	-0.296	-0.680	-1.063	-1.254
Bottom	1.136	4.140	4.165	4.445	4.833	5.221	5.609	5.803
Self wt.								
Precast-top	0.000	0.107	0.150	0.596	1.091	1.446	1.658	1.729
Bottom	-0.000	-0.108	-0.152	-0.603	-1.105	-1.463	-1.678	-1.750
DL-Prec (DC)								
Precast-top	0.000	0.036	0.051	0.202	0.370	0.491	0.563	0.587
Bottom	-0.000	-0.037	-0.052	-0.205	-0.375	-0.496	-0.569	-0.594

		Bearing	Trans	H/2	0.10L /0.90L	0.20L /0.80L	0.30L /0.70L	0.40L /0.60L	Midspan
DL-Prec (DW)									
Precast-top		-0.000	-0.000	-0.000	-0.000	-0.000	-0.000	-0.000	-0.000
Bottom		-0.000	-0.000	-0.000	-0.000	-0.000	-0.000	-0.000	-0.000
Diaphragm									
Precast-top		-0.000	0.001	0.001	0.004	0.008	0.011	0.012	0.014
Bottom		-0.000	-0.001	-0.001	-0.004	-0.008	-0.011	-0.012	-0.014
Deck + Haunch									
Precast-top		0.000	0.081	0.114	0.452	0.829	1.098	1.260	1.314
Bottom		-0.000	-0.082	-0.115	-0.458	-0.839	-1.112	-1.275	-1.330
DL-Comp (DC)									
Precast-top		-0.000	0.005	0.007	0.027	0.050	0.066	0.076	0.079
Bottom		0.000	-0.011	-0.015	-0.059	-0.108	-0.143	-0.164	-0.171
DL-Comp (DW)									
Precast-top		0.000	0.018	0.025	0.101	0.184	0.244	0.280	0.292
Bottom		-0.000	-0.039	-0.055	-0.218	-0.399	-0.529	-0.606	-0.632
LL+I(+)									
Precast-top		-0.000	0.060	0.084	0.331	0.603	0.792	0.903	0.935
Bottom		0.000	-0.129	-0.181	-0.716	-1.305	-1.714	-1.957	-2.026
Final 1 (P/S + DL	+ LL)								
Precast-top		0.125	0.695	0.795	1.799	2.839	3.467	3.689	3.695
Bottom		1.136	3.734	3.595	2.183	0.694	-0.247	-0.653	-0.714
Final 2 (P/S + DL)									
Precast-top		0.125	0.636	0.711	1.469	2.237	2.676	2.786	2.759
Bottom		1.136	3.863	3.776	2.899	1.999	1.467	1.303	1.313



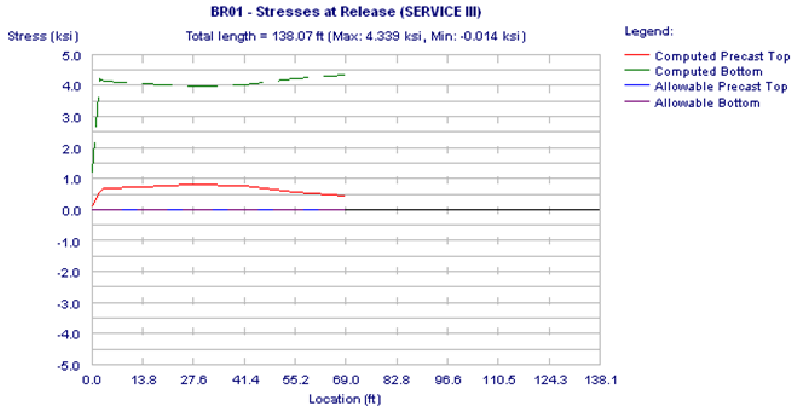
BR01 - Positive Stresses at Final 1, Span 1, Beam 1, SERVICE I

Span : 1, Beam : 1, SERVICE III

RELEASE STRESSES, (ksi) (LOSS = 10.71 %)

	Trans	0.10L /0.90L	0.20L /0.80L	0.30L /0.70L	0.40L /0.60L	Midspan	Depress.
Location, ft	3.00	13.97	27.95	41.92	55.90	69.87	62.88
Beam-Self							
Precast-top	0.149	0.638	1.133	1.488	1.700	1.771	1.753
Bottom	-0.151	-0.645	-1.147	-1.506	-1.721	-1.793	-1.775
Prestress							
Precast-top	0.410	0.092	-0.313	-0.718	-1.123	-1.326	-1.326
Bottom	4.375	4.697	5.107	5.517	5.927	6.132	6.132
Total							
Precast-top	0.559	0.730	0.820	0.770	0.577	0.446	0.428

	Trans	0.10L /0.90L	0.20L /0.80L	0.30L /0.70L	0.40L /0.60L	Midspan	Depress.
Bottom	4.224	4.051	3.959	4.011	4.206	4.339	4.357
As_top, in2	0.000	0.000	0.000	0.000	0.000	0.000	0.000
Ast_prvd, in2	0.000	0.000	0.000	0.000	0.000	0.000	0.000



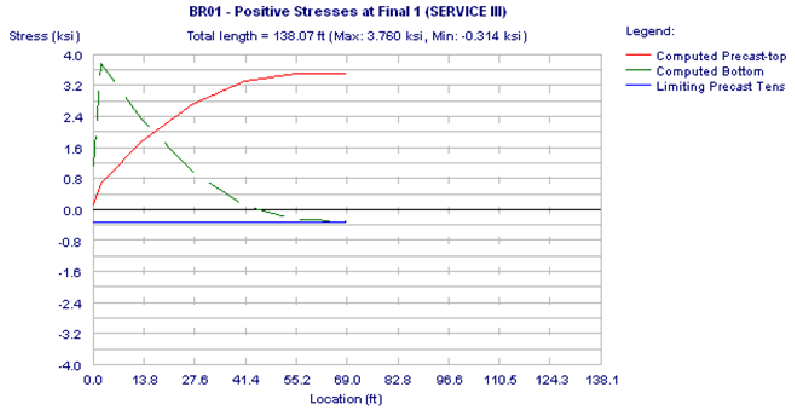
BR01 - Stresses at Release, Span 1, Beam 1, SERVICE III

SERVICE III

POSITIVE ENVELOPE STRESSES, (ksi) (LOSS = 15.50 %)

		Bearing	Trans	H/2	0.10L /0.90L	0.20L /0.80L	0.30L /0.70L	0.40L /0.60L	Midspan
Location, ft		0.00	2.17	3.06	13.14	27.11	41.09	55.06	69.04
Prestress									
Precast-top		0.125	0.388	0.363	0.087	-0.296	-0.680	-1.063	-1.254

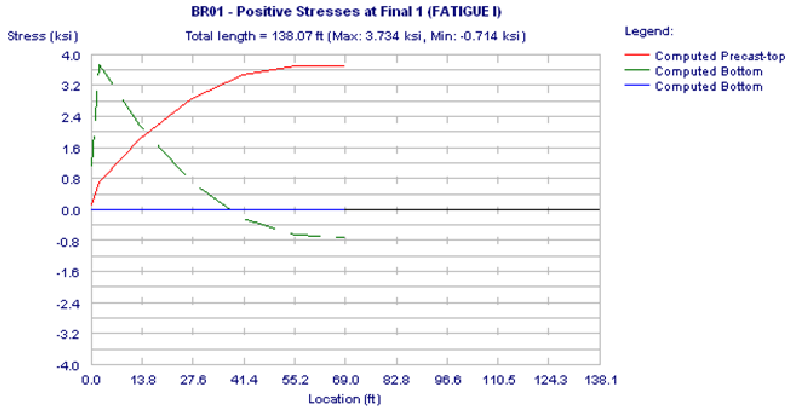
		Bearing	Trans	H/2	0.10L /0.90L	0.20L /0.80L	0.30L /0.70L	0.40L /0.60L	Midspan
Bottom		1.136	4.140	4.165	4.445	4.833	5.221	5.609	5.803
Self wt.									
Precast-top		0.000	0.107	0.150	0.596	1.091	1.446	1.658	1.729
Bottom		-0.000	-0.108	-0.152	-0.603	-1.105	-1.463	-1.678	-1.750
DL-Prec (DC)									
Precast-top		0.000	0.036	0.051	0.202	0.370	0.491	0.563	0.587
Bottom		-0.000	-0.037	-0.052	-0.205	-0.375	-0.496	-0.569	-0.594
DL-Prec (DW)									
Precast-top		-0.000	-0.000	-0.000	-0.000	-0.000	-0.000	-0.000	-0.000
Bottom		-0.000	-0.000	-0.000	-0.000	-0.000	-0.000	-0.000	-0.000
Diaphragm									
Precast-top		-0.000	0.001	0.001	0.004	0.008	0.011	0.012	0.014
Bottom		-0.000	-0.001	-0.001	-0.004	-0.008	-0.011	-0.012	-0.014
Deck + Haunch									
Precast-top		0.000	0.081	0.114	0.452	0.829	1.098	1.260	1.314
Bottom		-0.000	-0.082	-0.115	-0.458	-0.839	-1.112	-1.275	-1.330
DL-Comp (DC)									
Precast-top		-0.000	0.005	0.007	0.027	0.050	0.066	0.076	0.079
Bottom		0.000	-0.011	-0.015	-0.059	-0.108	-0.143	-0.164	-0.171
DL-Comp (DW)									
Precast-top		0.000	0.018	0.025	0.101	0.184	0.244	0.280	0.292
Bottom		-0.000	-0.039	-0.055	-0.218	-0.399	-0.529	-0.606	-0.632
LL+I(+)									
Precast-top		-0.000	0.048	0.067	0.265	0.482	0.633	0.723	0.748
Bottom		0.000	-0.103	-0.145	-0.573	-1.044	-1.372	-1.565	-1.621
Final 1 (P/S + DL	+ LL)								
Precast-top		0.125	0.683	0.778	1.733	2.719	3.309	3.508	3.508
Bottom		1.136	3.760	3.631	2.326	0.955	0.096	-0.262	-0.308



BR01 - Positive Stresses at Final 1, Span 1, Beam 1, SERVICE III

Span : 1, Beam : 1, FATIGUE I
POSITIVE ENVELOPE STRESSES, (ksi)

		Bearing	Trans	H/2	0.10L /0.90L	0.20L /0.80L	0.30L /0.70L	0.40L /0.60L	Midspan
Location, ft		0.00	2.17	3.06	13.14	27.11	41.09	55.06	69.04
F_LL+I(+)									
Precast-top		-0.000	0.040	0.056	0.219	0.393	0.516	0.582	0.588
Bottom		-0.000	-0.086	-0.121	-0.475	-0.852	-1.117	-1.261	-1.274
Final 3 (50% P/S	+ 50%	DL + F_LL)							
Precast-top		0.062	0.358	0.412	0.953	1.512	1.853	1.975	1.968
Bottom		0.568	1.845	1.767	0.975	0.148	-0.383	-0.609	-0.617



BR01 - Positive Stresses at Final 1, Span 1, Beam 1, FATIGUE I

VERTICAL/HORIZONTAL SHEAR

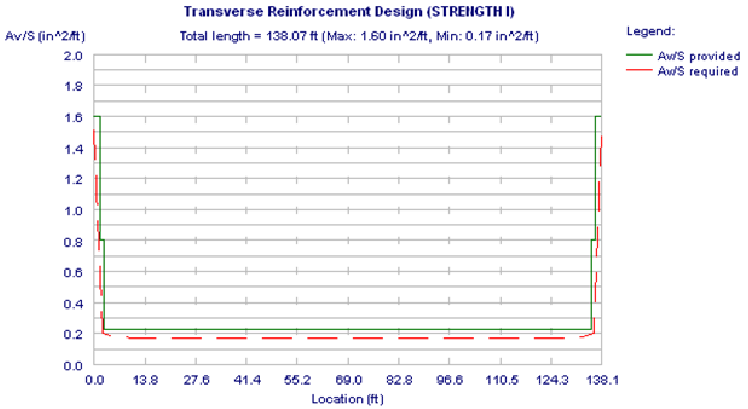
VERTICAL SHEAR (Art. 5.8) - Span : 1, Beam : 1, STRENGTH I
Using General Beta Theta Equation procedure - Art.5.8.3.4.2

Location(ft) / Mcor (kft)	Vu (kips) / a (in)	bv (in) / dv (in)	de (in) / fpo (ksi)	Aps (in2) / vu/fc	Vp (kips) / Vc-com (kips)	eps_x / Beta	Theta / Max.spc. (in)	Vs-reqd (kips) / min.Av/s (in2/ft)	Av/s (in2/ft) / pVn/Vu	Av-prvd (in2/ft) / Aps* (in2)	Al_reqd (in2)
Bearing :	0.83										
433.9	8.00	67.69	1.841	10.5	6.00e-3	50.0	422.7	1.507	1.600	0.00	
0.0	1.63	66.87	52.6	0.080	48.9	0.87	24.00	0.168	1.054	1.662	
Transfer :	3.00										
422.2	8.00	67.69	9.331	37.6	-0.21e-3	28.3	122.5	0.203	0.800	0.00	
906.8	5.77	64.80	189.0	0.076	309.1	5.69	24.00	0.168	1.766	5.970	
Critical :	6.20										
404.8	8.00	67.69	9.331	37.6	-0.21e-3	28.3	105.5	0.176	0.229	0.00	
2124.4	6.69	64.34	189.0	0.073	306.7	5.68	24.00	0.168	1.069	7.129	
0.1L :	13.97										
362.6	8.00	67.69	9.331	37.6	-0.10e-3	28.6	89.2	0.168	0.229	0.00	
5003.9	8.74	63.32	189.0	0.066	276.1	5.20	24.00	0.168	1.107	9.331	
0.2L :	27.95										
289.3	8.00	62.75	11.284	37.6	-0.01e-3	29.0	47.9	0.168	0.229	0.00	
9014.2	8.79	58.36	189.0	0.055	235.9	4.82	24.00	0.168	1.226	11.284	
0.3L :	41.92										
216.7	8.00	62.72	12.586	37.6	0.59e-3	31.1	40.1	0.168	0.229	0.00	
11713.8	8.80	58.32	189.0	0.040	163.1	3.33	24.00	0.168	1.293	12.586	
0.4L :	55.90										
145.7	8.00	65.66	12.586	37.6	0.86e-3	32.0	0.0	0.168	0.229	0.00	
13157.0	8.81	61.25	189.0	0.023	150.2	2.92	24.00	0.168	1.852	12.586	
0.5L :	69.87										
76.2	8.00	67.12	12.586	0.0	0.73e-3	31.6	0.0	0.168	0.229	0.00	
13411.6	8.81	62.72	189.0	0.015	162.8	3.09	24.00	0.168	3.301	12.586	
0.6L :	83.84										
145.7	8.00	65.66	12.586	37.6	0.86e-3	32.0	0.0	0.168	0.229	0.00	
13157.0	8.81	61.25	189.0	0.023	150.2	2.92	24.00	0.168	1.852	12.586	
0.7L :	97.82										

Location(ft) / Mcor (kft)	Vu (kips) / a (in)	bv (in) / dv (in)	de (in) / fpo (ksi)	Aps (in2) / vu/fc	Vp (kips) / Vc-com (kips)	eps_x / Beta	Theta / Max.spc. (in)	Vs-reqd (kips) / min.Av/s (in2/ft)	Av/s (in2/ft) / pVn/Vu	Av-prvd (in2/ft) / Aps* (in2)	Al_reqd (in2)
216.7	8.00	62.72	12.586	37.6	0.59e-3	31.1	40.1	0.168	0.229	0.00	
11713.8	8.80	58.32	189.0	0.040	163.1	3.33	24.00	0.168	1.293	12.586	
0.8L :	111.79										
289.3	8.00	62.75	11.284	37.6	-0.01e-3	29.0	47.9	0.168	0.229	0.00	
9014.2	8.79	58.36	189.0	0.055	235.9	4.82	24.00	0.168	1.226	11.284	
0.9L :	125.77										
362.6	8.00	67.69	9.331	37.6	-0.10e-3	28.6	89.2	0.168	0.229	0.00	
5003.9	8.74	63.32	189.0	0.066	276.1	5.20	24.00	0.168	1.107	9.331	
Critical :	133.54										
404.8	8.00	67.69	9.331	37.6	-0.21e-3	28.3	105.5	0.176	0.229	0.00	
2124.4	6.69	64.34	189.0	0.073	306.7	5.68	24.00	0.168	1.069	7.129	
Transfer :	136.74										
422.2	8.00	67.69	9.331	37.6	-0.21e-3	28.3	122.5	0.203	0.800	0.00	
906.8	5.77	64.80	189.0	0.076	309.1	5.69	24.00	0.168	1.766	5.970	
Bearing :	138.90										
433.9	8.00	67.69	1.841	10.5	6.00e-3	50.0	422.7	1.507	1.600	0.00	
0.0	1.63	66.87	52.6	0.080	48.9	0.87	24.00	0.168	1.054	1.662	

ANCHORAGE ZONE REINFORCEMENT (Art. 5.10.10)
Span : 1, Beam : 1

Fpi (kips)	fs (ksi)	h/4 (in)	Abrst_rqrd (in2)
2548.66	20.00	16.50	5.10



BR01 - Vertical Shear, Span 1, Beam 1, STRENGTH I

HORIZONTAL SHEAR (Art. 5.8.4) - Span : 1, Beam : 1
(Beam and Slab effects are INCLUDED in Vu).
Computed Interface width considered to be engaged in shear transfer, bvi = 48.00(in).

Location (ft) / Vu (kips)	Vnh-req (kips/in)	de (in)	a (in)	dv (in)	s_max (in)	Avh-min (in2/ft)	Avh-sm (in2/ft)	Avh-rg (in2/ft)	Avh-prvd (in2/ft)
Bearing : 433.9	0.00 7.21	67.69	1.63	66.87	24.00	0.480	1.203	0.000	1.600
Transfer : 422.2	2.17 7.24	67.69	5.77	64.80	24.00	0.480	1.213	0.000	0.800*
Critical : 404.8	5.36 6.99	67.69	6.69	64.34	24.00	0.480	1.130	0.000	0.229*
0.1L :	13.14								

SAY OK, SURFACE IS ROUGHENED

Location (ft) / Vu (kips)	Vnh-req (kips/in)	de (in)	a (in)	dv (in)	s_max (in)	Avh-min (in2/ft)	Avh-sm (in2/ft)	Avh-rg (in2/ft)	Avh-prvd (in2/ft)
362.6	6.36	67.69	8.74	63.32	24.00	0.480	0.921	0.000	0.229*
0.2L : 289.3	27.11 5.51	62.75	8.79	58.36	24.00	0.480	0.636	0.000	0.229*
0.3L : 216.7	41.09 4.13	62.72	8.80	58.32	24.00	0.480	0.176	0.000	0.229
0.4L : 145.7	55.06 2.64	65.66	8.81	61.25	24.00	0.480	0.000	0.000	0.229
0.5L : 76.2	69.04 1.35	67.12	8.81	62.72	24.00	0.480	0.000	0.000	0.229
0.6L : 145.7	83.01 2.64	65.66	8.81	61.25	24.00	0.480	0.000	0.000	0.229
0.7L : 216.7	96.98 4.13	62.72	8.80	58.32	24.00	0.480	0.176	0.000	0.229
0.8L : 289.3	110.96 5.51	62.75	8.79	58.36	24.00	0.480	0.636	0.000	0.229*
0.9L : 362.6	124.93 6.36	67.69	8.74	63.32	24.00	0.480	0.921	0.000	0.229*
Critical : 404.8	132.71 6.99	67.69	6.69	64.34	24.00	0.480	1.130	0.000	0.229*
Transfer : 422.2	135.90 7.24	67.69	5.77	64.80	24.00	0.480	1.213	0.000	0.800*
Bearing : 433.9	138.07 7.21	67.69	1.63	66.87	24.00	0.480	1.203	0.000	1.600

SAY OK, SURFACE IS ROUGHENED

CAMBER/DEFLECTION

CAMBER AND DEFLECTIONS: SERVICE I
(Span : 1, Beam : 1; Units: in)

	Release	Mult	Erection	Mult	Final
At 0.1 x L =	13.14 ft				
Prestress	2.186	1.80	3.935	2.20	4.810
Self Wt.	-0.946	1.85	-1.749	2.40	-2.270
Deck + Haunch			-0.539	2.30	-1.240
DL-Prec. (DC)			-0.241	3.00	-0.722
Diaphragm			-0.005	3.00	-0.016
DL-Prec. (DW)			0.000	3.00	0.000
DL-Comp. (DC)			-0.051	3.00	-0.153
DL-Comp. (DW)			-0.188	3.00	-0.565
Live Load					-0.339
Total	1.241		1.161		-0.494

	Release	Mult	Erection	Mult	Final
At 0.2 x L =	27.11 ft				
Prestress	4.002	1.80	7.204	2.20	8.805
Self Wt.	-1.789	1.85	-3.310	2.40	-4.294
Deck + Haunch			-1.053	2.30	-2.422
DL-Prec. (DC)			-0.470	3.00	-1.411
Diaphragm			-0.010	3.00	-0.031
DL-Prec. (DW)			0.000	3.00	0.000
DL-Comp. (DC)			-0.099	3.00	-0.298
DL-Comp. (DW)			-0.368	3.00	-1.105
Live Load					-0.667
Total	2.213		1.893		-1.423

	Release	Mult	Erection	Mult	Final
At 0.3 x L =	41.09 ft				
Prestress	5.380	1.80	9.683	2.20	11.835
Self Wt.	-2.449	1.85	-4.531	2.40	-5.879
Deck + Haunch			-1.457	2.30	-3.350
DL-Prec. (DC)			-0.650	3.00	-1.951
Diaphragm			-0.014	3.00	-0.043
DL-Prec. (DW)			0.000	3.00	0.000
DL-Comp. (DC)			-0.138	3.00	-0.413
DL-Comp. (DW)			-0.509	3.00	-1.528
Live Load					-0.933
Total	2.930		2.383		-2.262

	Release	Mult	Erection	Mult	Final
At 0.4 x L =	55.06 ft				

	Release	Mult	Erection	Mult	Final
Prestress	6.251	1.80	11.252	2.20	13.753
Self Wt.	-2.869	1.85	-5.307	2.40	-6.885
Deck + Haunch			-1.713	2.30	-3.941
DL-Prec. (DC)			-0.765	3.00	-2.295
Diaphragm			-0.017	3.00	-0.050
DL-Prec. (DW)			0.000	3.00	0.000
DL-Comp. (DC)			-0.162	3.00	-0.485
DL-Comp. (DW)			-0.599	3.00	-1.797
Live Load					-1.106
Total	3.382		2.689		-2.807

	Release	Mult	Erection	Mult	Final
At 0.5 x L =	69.04 ft				
Prestress	6.552	1.80	11.794	2.20	14.415
Self Wt.	-3.012	1.85	-5.573	2.40	-7.230
Deck + Haunch			-1.801	2.30	-4.143
DL-Prec. (DC)			-0.804	3.00	-2.413
Diaphragm			-0.018	3.00	-0.053
DL-Prec. (DW)			0.000	3.00	0.000
DL-Comp. (DC)			-0.170	3.00	-0.510
DL-Comp. (DW)			-0.630	3.00	-1.889
Live Load					-1.159
Total	3.540		2.798		-2.982

	Release	Mult	Erection	Mult	Final
At 0.6 x L =	83.01 ft				
Prestress	6.251	1.80	11.252	2.20	13.753
Self Wt.	-2.869	1.85	-5.307	2.40	-6.885
Deck + Haunch			-1.713	2.30	-3.941
DL-Prec. (DC)			-0.765	3.00	-2.295
Diaphragm			-0.017	3.00	-0.050
DL-Prec. (DW)			0.000	3.00	0.000
DL-Comp. (DC)			-0.162	3.00	-0.485
DL-Comp. (DW)			-0.599	3.00	-1.797
Live Load					-1.106
Total	3.382		2.689		-2.807

	Release	Mult	Erection	Mult	Final
At 0.7 x L =	96.98 ft				
Prestress	5.380	1.80	9.683	2.20	11.835
Self Wt.	-2.449	1.85	-4.531	2.40	-5.879
Deck + Haunch			-1.457	2.30	-3.350
DL-Prec. (DC)			-0.650	3.00	-1.951
Diaphragm			-0.014	3.00	-0.043
DL-Prec. (DW)			0.000	3.00	0.000
DL-Comp. (DC)			-0.138	3.00	-0.413
DL-Comp. (DW)			-0.509	3.00	-1.528

	Release	Mult	Erection	Mult	Final
Live Load					-0.933
Total	2.930		2.383		-2.262

	Release	Mult	Erection	Mult	Final
At 0.8 x L =	110.96 ft				
Prestress	4.002	1.80	7.204	2.20	8.805
Self Wt.	-1.789	1.85	-3.310	2.40	-4.294
Deck + Haunch			-1.053	2.30	-2.422
DL-Prec. (DC)			-0.470	3.00	-1.411
Diaphragm			-0.010	3.00	-0.031
DL-Prec. (DW)			0.000	3.00	0.000
DL-Comp. (DC)			-0.099	3.00	-0.298
DL-Comp. (DW)			-0.368	3.00	-1.105
Live Load					-0.667
Total	2.213		1.893		-1.423

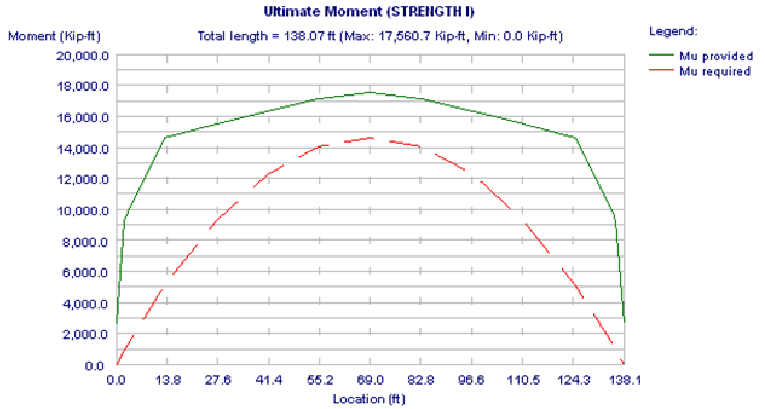
	Release	Mult	Erection	Mult	Final
At 0.9 x L =	124.93 ft				
Prestress	2.186	1.80	3.935	2.20	4.810
Self Wt.	-0.946	1.85	-1.749	2.40	-2.270
Deck + Haunch			-0.539	2.30	-1.240
DL-Prec. (DC)			-0.241	3.00	-0.722
Diaphragm			-0.005	3.00	-0.016
DL-Prec. (DW)			0.000	3.00	0.000
DL-Comp. (DC)			-0.051	3.00	-0.153
DL-Comp. (DW)			-0.188	3.00	-0.565
Live Load					-0.339
Total	1.241		1.161		-0.494

ULTIMATE MOMENT

ULTIMATE - Span : 1, Beam : 1, STRENGTH I
(Mr-prvd computed by Strain Compatibility method. Ult. Conc. Strain = 0.00300)

Location (ft) Mu k.ft	dp in	Aps in2	fps ksi	c in	a in	Mr-prvd k.ft	c/dt	Phi	1.2 Mcr k.ft	min Mr k.ft	Crkg Ratio	Mu-p/r Ratio
Transfer	2.17											
912.3	55.0	8.053	265.3	7.0	5.8	9282.1	0.097T	1.00	-	-	-	-
H/2	3.06											
1281.8	55.2	8.492	265.2	7.3	6.1	9789.2	0.103T	1.00	-	-	-	-
0.1L	13.14											
5081.5	57.2	12.586	265.1	10.6	8.7	14643.3	0.148T	1.00	11427.0	6758.4	1.5	-
0.2L	27.11											
9290.5	59.9	12.586	266.6	10.6	8.8	15480.7	0.148T	1.00	11723.9	11723.9	1.6	-
0.3L	41.09											
12261.9	62.7	12.586	267.1	10.6	8.8	16313.5	0.149T	1.00	12171.7	12171.7	1.6	-
0.4L	55.06											
14033.7	65.7	12.586	267.4	10.6	8.8	17145.1	0.149T	1.00	12770.6	12770.6	1.6	-
0.5L	69.04											
14591.2	67.1	12.586	267.4	10.6	8.8	17560.7	0.149T	1.00	13107.2	13107.2	1.6	-
0.6L	83.01											
14033.7	65.7	12.586	267.4	10.6	8.8	17145.1	0.149T	1.00	12770.6	12770.6	1.6	-
0.7L	96.98											
12261.9	62.7	12.586	267.1	10.6	8.8	16313.5	0.149T	1.00	12171.7	12171.7	1.6	-
0.8L	110.96											
9290.5	59.9	12.586	266.6	10.6	8.8	15480.7	0.148T	1.00	11723.9	11723.9	1.6	-
0.9L	124.93											
5081.5	57.2	12.586	265.1	10.6	8.7	14643.3	0.148T	1.00	11427.0	6758.4	1.5	-
H/2	135.01											
1281.8	55.2	8.492	265.2	7.3	6.1	9789.2	0.103T	1.00	-	-	-	-
Transfer	135.90											
912.3	55.0	8.053	265.3	7.0	5.8	9282.1	0.097T	1.00	-	-	-	-

Legend: C = Compression-Controlled (c/dt > 0.600)
I = In-Transition (0.60 >= c/dt > 0.375)
T = Tension-Controlled (c/dt <= 0.375)
Note : fr used for calculating Mcr is computed using AASHTO method (Art.5.4.2.6.)
Consider Bottom Tension Steel Contribution : NO



BR01 - Ultimate Moment, Span 1, Beam 1, STRENGTH I

DETENSIONING

Span : 1, Beam : 1; Groups 1-32; Units: ksi

Grp	Str		Ys,in		3.00ft
1	2	E	2.00	Ft	0.082
		M	2.00	Fb	0.082
2	2	E	63.00	Ft	0.295
		M	12.00	Fb	0.032
3	1	E	63.00	Ft	0.401
		M	12.00	Fb	0.007
4	2	E	61.00	Ft	0.604
		M	10.00	Fb	-0.033
5	1	E	61.00	Ft	0.706
		M	10.00	Fb	-0.054
6	2	E	59.00	Ft	0.899
		M	8.00	Fb	-0.084
7	1	E	59.00	Ft	0.996
		M	8.00	Fb	-0.099
8	2	E	57.00	Ft	1.180
		M	6.00	Fb	-0.120
9	1	E	57.00	Ft	1.272
		M	6.00	Fb	-0.131
10	2	E	55.00	Ft	1.446
		M	4.00	Fb	-0.142
11	1	E	55.00	Ft	1.533
		M	4.00	Fb	-0.148
12	2	E	12.00	Ft	1.514
		M	12.00	Fb	0.036
13	2	E	10.00	Ft	1.486
		M	10.00	Fb	0.230
14	2	E	10.00	Ft	1.457
		M	10.00	Fb	0.424
15	2	E	10.00	Ft	1.429
		M	10.00	Fb	0.618
16	2	E	8.00	Ft	1.391
		M	8.00	Fb	0.822
17	2	E	8.00	Ft	1.353
		M	8.00	Fb	1.026
18	2	E	8.00	Ft	1.315
		M	8.00	Fb	1.229
19	2	E	8.00	Ft	1.277
		M	8.00	Fb	1.433
20	2	E	6.00	Ft	1.229
		M	6.00	Fb	1.646
21	2	E	6.00	Ft	1.182
		M	6.00	Fb	1.859
22	2	E	6.00	Ft	1.134
		M	6.00	Fb	2.073

Grp	Str		Ys,in		3.00ft
23	2	E	6.00	Ft	1.087
		M	6.00	Fb	2.286
24	2	E	4.00	Ft	1.030
		M	4.00	Fb	2.509
25	2	E	4.00	Ft	0.973
		M	4.00	Fb	2.732
26	2	E	4.00	Ft	0.916
		M	4.00	Fb	2.955
27	2	E	4.00	Ft	0.859
		M	4.00	Fb	3.178
28	2	E	2.00	Ft	0.792
		M	2.00	Fb	3.410
29	2	E	2.00	Ft	0.725
		M	2.00	Fb	3.643
30	2	E	2.00	Ft	0.659
		M	2.00	Fb	3.875
31	2	E	2.00	Ft	0.592
		M	2.00	Fb	4.108
32	1	E	2.00	Ft	0.559
		M	2.00	Fb	4.224

DESIGN SUMMARY

Span: 1, Beam: 1, Exterior beam

Beam type:	I-Girder,	ODOT 66" - 4ft - SHOP
Precast Length,	ft	139.74
Release Length,	ft	139.74
Strand Pattern:	Straight/Draped	Depr. Point: 0.45 L
Strand:	6/10-270K-LL	
Strand Es,	ksi:	28500.0
No. of strands:	58	
	Draped:	15
	Straight:	43
Concrete Strength:		
	f'ci:	7.5 ksi
	f'c:	11.0 ksi
	f'ct:	4.5 ksi
Initial losses:	10.71 %	
Final losses:	15.50 %	

Specification	Allowable	Computed	Location	Status
Release Stresses (ksi) (Art. 5.9.4.1)				
Precast Bot (compression)	4.500	4.357	Depress	OK
Precast Top w/ no reinf. (tension)	-0.200	0.446	Midspan	
Precast Top w/ reinf. (tension)	-0.657			
Strength I (Art. 3.4.1, 5.7.3.1.1)	Provided	Required	Location	Status
Ult. Moment (k.ft)	17560.70	14591.25	Midspan	OK
Debonding Limits (Art. 5.11.4.3)	Allowable	Computed		Status
Max. Debond per Row	40.00 %	0.00 %		OK
Max. Debond Total	25.00 %	0.00 %		OK

Positive Moment Envelope Stresses (ksi) (Art. 3.4.1 and 5.9.4.2)

Specification	Allow	Final 1 Comp	Loc.	Allow	Final 2 Comp	Loc.	Allow	Final 3 Comp	Loc.
Service I Limit State - Compressive	Stresses	Only							
Precast Top	6.600	3.695	Midspan	4.950	2.786	0.4L/0.6L			
Precast Bot	6.600	3.734	Transfer	4.950	3.863	Transfer			
Service III Limit State - Tensile	Stresses	Only							
Precast Top	-0.314	0.125	Bearing						
Precast Bot	-0.314	-0.308	Midspan						
Fatigue I Limit State - Compressive	Stresses	Only							
Precast Top							4.400	1.975	0.4L/0.6L
Precast Bot							4.400	1.845	Transfer

CAMBER / DEFLECTION: (PCI Design Handbook - 4th Ed.- Table 4.6.2)
0.5 x L = 69.04 ft

	Release	Mult	Erection	Mult	Final
Prestress	6.552	1.80	11.794	2.20	14.415
Self Wt.	-3.012	1.85	-5.573	2.40	-7.230
Deck + Haunch			-1.801	2.30	-4.143
DL-Prec. (DC)			-0.804	3.00	-2.413
Diaphragm			-0.018	3.00	-0.053
DL-Prec. (DW)			0.000	3.00	0.000
DL-Comp. (DC)			-0.170	3.00	-0.510
DL-Comp. (DW)			-0.630	3.00	-1.889
Live Load					-1.159
Total	3.540		2.798		-2.982

Positive values indicate upward deflection.

PROPERTIES

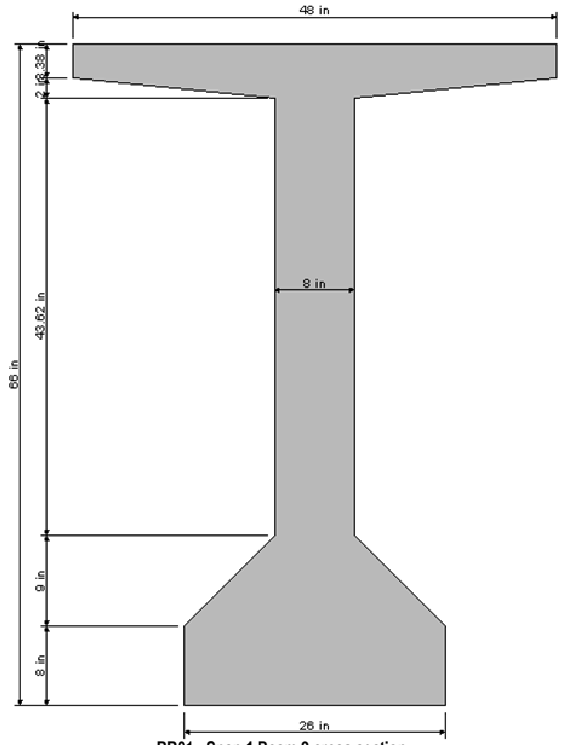
Span:1, Beam:2

PRECAST DATA:

Section Id	ODOT 66" - 4ft - SHOP					
Type	I-Girder					
Flng width	Top	48.000	in	Bot	26.000	in
thick	Top	3.380	in	Bot	8.000	in
Stems	No	1				
	Top	8.000	in			
	Bot	8.000	in			
Shear width		8.000	in			

Article 5.14.1.2.2 checked: OK.

Bentley			Sheet #	DS-2
			Job #	49633
Program:	LEAP® CONSPAN® V8i (SELECTseries 2)	Great Lakes Client Licenses	Designed By	AKS
Version:	10.00.02.19	Copyright © Bentley Systems, Inc. 2011	Date	Apr/18/2011
		www.bentley.com Phone: 1-800-778-4277	Checked	DBT
File Name:	**CUY-90-1627-date120210-FINAL.csl**		Date	Apr/18/2011



BR01 - Span 1 Beam 2 cross section

GENERAL BRIDGE DATA:

Bridge Width	86.32	ft
Curb-to-curb	83.32	ft
Beam Spac. Lt./Rt	8.83/ 8.83	ft
Lane width	12.00	ft
Number of lanes	7	
Interior/Exterior	Interior	
Start Skew Angle	12.12	degrees
End Skew Angle	12.12	degrees

TOPPING DATA:

Deck	Thickness	7.500	in	
Haunch:				
	Thickness	0.000	in	
	Width	48.000	in	
Effective	width	106.000	in	(Art. 4.6.2.6.1)

GENERAL LOAD DATA:
DEAD LOADS ON PRECAST
UNITS: (Point: kips, Location: ft, Line: klf, Trapez: klf)

DC/DW	Type	Mag.1	Loc.1	Mag.2	Loc.2	Description
DC	Line	0.150	0.000	0.150	138.070	Haunch
DC	Line	0.110	0.000	0.110	138.070	Sacrificial Wearing Surface

Diaphragm loads:
(kips, ft)

Mag.	Loc.
0.55	34.52
0.55	69.03
0.55	103.55

Dead loads on composite: See Project info for composite loads

GENERAL SPAN DATA:

Overall length	139.740	ft
Release length	139.740	ft
Design length	138.070	ft

KERN POINTS:

Upper	50.22	in
Lower	15.97	in

DISTRIBUTION FACTORS (Art. 4.6.2.2):
Type k, with deck

Live Moment	(2+ lanes loaded)	0.710	(Calculated)
Live Moment	(1 lane loaded)	0.478	(Calculated)
Live Shear	(2+ lanes loaded)	0.910	(Calculated)
Live Shear	(1 lane loaded)	0.734	(Calculated)

Pedestrian	0.102	(Calculated)
Comp. DC	0.102	(Calculated)
Comp. DW	0.102	(Calculated)

Dead Loads and Pedestrian Load distributed based on Tributary Fraction
RESISTANCE FACTORS (Art. 5.5.4.2):

Flexure Reinforced	
Compression controlled sections	0.75
Tension controlled sections	0.90
Flexure Prestressed	
Compression controlled sections	0.75
Tension controlled sections	1.00
Shear	0.90

SECTION PROPERTIES:

	PRECAST		COMPOSITE		
Area	956.0	in2	1464.5	in2	#
Total Height	66.00	in	73.50	in	
Mom. of Inertia (Ixx)	540204	in4	986017	in4	#
Ht. of c.g.	33.20	in	45.89	in	#
Density	150.00	pcf	150.00	pcf	
Self-weight	995.8	plf	1824.0	plf	
Mom. of Inertia (Iyy)	58471.3	in4			
Poisson's Ratio	0.2				

(#) Of Total Section using Ect/Ec = 0.6396
Use transformed strand and rebar: No

Span:1, Beam:2
STRESS LIMITS (Art. 5.9.4):
STRESS LIMITS AT RELEASE BEFORE LOSSES:

		PRECAST	
Strength		7.50	ksi
Elasticity		5250.3	ksi
Max comp		4.50	ksi
Max tens		-0.20	ksi
Max tens,	w/reinf	-0.66	ksi

STRESS LIMITS AT FINAL AFTER LOSSES:

	PRECAST		DECK	
Strength	11.00	ksi	4.50	ksi
Elasticity	6358.39	ksi	4066.84	ksi

STRESS LIMITS AT FINAL 1 (P/S + DL + LL):

	PRECAST		DECK	
Max comp	6.60	ksi	2.70	ksi

STRESS LIMITS AT FINAL 2 (P/S + DL):

	PRECAST		DECK	
Max comp	4.95	ksi	2.03	ksi

FATIGUE I STRESS LIMITS AT FINAL 3 (50% P/S + 50% DL + F_LL):

	PRECAST		DECK	
Max comp	4.40	ksi	-	ksi

SERVICE III (Tension):

	PRECAST		DECK	
Max tens	-0.31	ksi	-0.40	ksi

Span:1, Beam:2
PRESTRESSED STEEL:
58 strands, 6/10-270K-LL, Low relaxation strands
Depressed at 0.45L (62.88 ft from member end)

END PATTERN (Ycg = 19.57 in):

11 @ 2.000 in	8 @ 4.000 in	8 @ 6.000 in	8 @ 8.000 in
6 @ 10.000 in	2 @ 12.000 in	3 @ 55.000 in	3 @ 57.000 in
3 @ 59.000 in	3 @ 61.000 in	3 @ 63.000 in	

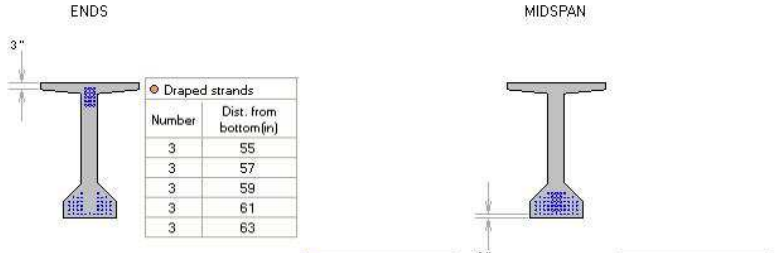
MID PATTERN (Ycg = 6.38 in):
(A) Draped:

3 @ 4.000 in	3 @ 6.000 in	3 @ 8.000 in	3 @ 10.000 in
3 @ 12.000 in			

(B) Straight:

11 @ 2.000 in	8 @ 4.000 in	8 @ 6.000 in	8 @ 8.000 in
6 @ 10.000 in	2 @ 12.000 in		

Strand Diameter	0.600	in
Strand Area	0.217	in2
Total Strand Area	12.586	in2
Trans. Len,bonded	3.000	ft
Trans. Len,debonded	3.000	ft
Dev. Len, bonded	12.307	ft
Dev. Len, debonded	15.384	ft
Holddown Force	44.447	kips
Tensile Strength(fpu)	270.0	ksi
Initial Prestress = 0.75fpu	202.5	ksi
Initial Pull	2548.7	kips
Beam Shrtng (PL/AE)	0.760	in



BR01 - Strand Pattern, Span 1, Beam 2

Span:1, Beam:2
ESTIMATED QUANTITIES

Prestressing (linear ft)	Strands (LB/1000ft)	(LB)	Beam Vol(C.Y.)	Concrete Wt(LB)	Stirrups (LB)	Longitudinal Bars (LB)
8109.223	740	6000.825	34.360	139158.047	797.368	0.000

Span:1, Beam:2
REINFORCING STEEL:

Tension	steel:	
fy	60.0	ksi
Es	29000	ksi
fs	24.0	ksi

Stirrups:

# legs	Size	fy (ksi)	Area (in2)	Spacing (in)	Start (ft)	End (ft)	Extends into Deck
2	US#4[M13]	60.0	0.40	3.00	0.0000	2.5000	Yes
2	US#4[M13]	60.0	0.40	6.00	2.5000	3.5000	Yes
2	US#4[M13]	60.0	0.40	21.00	3.5000	136.2402	Yes
2	US#4[M13]	60.0	0.40	6.00	136.2402	137.2402	Yes
2	US#4[M13]	60.0	0.40	3.00	137.2402	139.7402	Yes

LOSSES
Note: Values are calculated at Midspan

Str. area	12.5860	in2
Ycg	6.38	in
P_init	2548.7	kips
Ecc	26.82	in
Days to release	0.75	
Rel. Humid.(RH)	75.0	%
Es	28500.0	ksi
Eci	5250	ksi

AASHTO LOSSES

Elastic Shortening 21.69 ksi (Eq 5.9.5.2.3a-1), (fcgp= 3.995 ksi)

	Elastic Gains	Gains		Adjustment	
due	to Precast Loads	-7.03	ksi	0.80	ksi
due	to Composite Loads	-3.36	ksi	0.38	ksi
due	to Live Loads	-5.46	ksi	0.77	ksi

Time Dependent Losses (Approximate Method (Art.5.9.5.3))

		Initial			Final		
Steel relaxation	0.00	ksi		2.40	ksi	(Eq 5.9.5.3-1)
Concrete shrinkage	0.00	ksi		6.71	ksi	(Eq 5.9.5.3-1)
Concrete creep	0.00	ksi		14.90	ksi	(Eq 5.9.5.3-1)
Sub-total	21.69	ksi	(10.71 %)	10.11	ksi	(4.99 %)
Total Prestress Losses				31.80	ksi	(15.70 %)

Prestressing Stress Limit Check (Table 5.9.3.1)
initial fpi = 202.5 ksi < 0.75 fpu, OK
initial fpe = 170.7 ksi < 0.80 fpy, OK

SHEAR/MOMENT ENVELOPE (&REACTIONS)

SHEAR AND MOMENT ENVELOPE : Span : 1, Beam : 2, SERVICE I
Shears: kips, Moments: kft

		Bearing	Trans	H/2	0.10L	0.20L	0.30L	0.40L	Midspan
Location,	ft	0.00	2.17	3.06	13.14	27.11	41.09	55.06	69.04
Self wt. :	M	0.0	146.5	205.9	817.3	1497.9	1984.1	2275.8	2373.0
(Max)	V	68.7	66.6	65.7	55.7	41.7	27.8	13.9	0.0
DL-Prec. :	M	0.0	38.3	53.8	213.7	391.7	518.8	595.1	620.5
DC(Max)	V	18.0	17.4	17.2	14.6	10.9	7.3	3.6	0.0
DL-Prec. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DW(Max)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Deck + :	M	0.0	121.8	171.2	679.7	1245.7	1649.9	1892.5	1973.3
Haunch (Max)	V	57.2	55.4	54.6	46.3	34.7	23.1	11.6	0.0
Diaphragm :	M	0.0	1.8	2.5	10.8	22.4	30.3	34.1	38.0
(Max)	V	0.8	0.8	0.8	0.8	0.8	0.3	0.3	0.3
DL-Comp :	M	0.0	20.4	28.7	114.1	209.0	276.9	317.6	331.1
DC(Max)	V	9.6	9.3	9.2	7.8	5.8	3.9	1.9	0.0
DL-Comp :	M	0.0	75.7	106.4	422.3	773.9	1025.1	1175.8	1226.0
DW(Max)	V	35.5	34.4	33.9	28.8	21.6	14.4	7.2	0.0
LL + I :	M+	0.0	201.3	282.8	1119.0	2038.7	2678.1	3056.4	3165.1
	V	121.4	118.7	117.6	105.3	88.4	32.9	9.2	30.9
LL + I :	M-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
LL + I :	Vmx	121.4	118.7	117.6	105.3	89.7	74.9	60.8	47.7
	M	0.0	198.6	278.5	1079.9	1899.5	2402.0	2614.8	2570.8
Total :	M+	0.0	605.9	851.4	3376.9	6179.3	8163.1	9347.3	9727.1
	V	311.2	302.6	299.1	259.1	204.0	109.7	47.7	31.2
Total :	M-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total :	Vmx	311.2	302.6	299.1	259.1	205.4	151.7	99.4	48.0
	M	0.0	603.1	847.0	3337.8	6040.1	7887.0	8905.6	9132.9

		0.60L	0.70L	0.80L	0.90L	H/2	Trans	Bearing
Location,	ft	83.01	96.98	110.96	124.93	135.01	135.91	138.07
Self wt. :	M	2275.8	1984.1	1497.9	817.3	205.9	146.5	0.0
(Max)	V	13.9	27.8	41.7	55.7	65.7	66.6	68.7
DL-Prec. :	M	595.1	518.8	391.7	213.7	53.8	38.3	0.0
DC(Max)	V	3.6	7.3	10.9	14.6	17.2	17.4	18.0
DL-Prec. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DW(Max)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Deck + :	M	1892.5	1649.9	1245.7	679.7	171.2	121.8	0.0
Haunch (Max)	V	11.6	23.1	34.7	46.3	54.6	55.4	57.2
Diaphragm :	M	34.1	30.3	22.4	10.8	2.5	1.8	-0.0
(Max)	V	0.3	0.3	0.8	0.8	0.8	0.8	0.8
DL-Comp :	M	317.6	276.9	209.0	114.1	28.7	20.4	-0.0
DC(Max)	V	1.9	3.9	5.8	7.8	9.2	9.3	9.6
DL-Comp :	M	1175.8	1025.1	773.9	422.3	106.4	75.7	0.0
DW(Max)	V	7.2	14.4	21.6	28.8	33.9	34.4	35.5

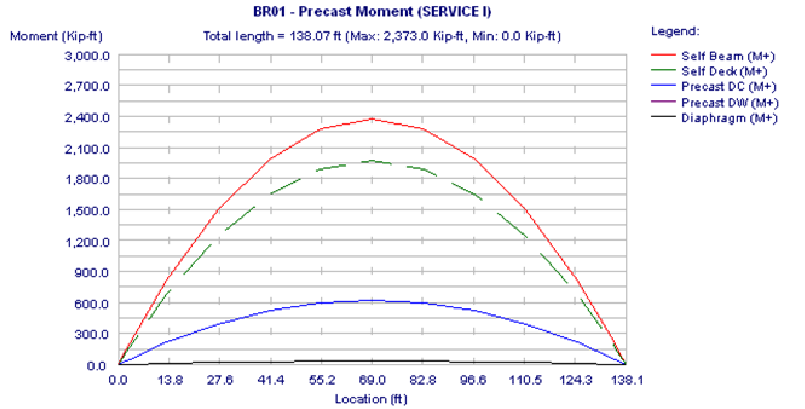
		0.60L	0.70L	0.80L	0.90L	H/2	Trans	Bearing
LL + I :	M+	3056.4	2678.1	2038.7	1119.0	282.8	201.3	-0.0
	V	9.2	32.9	88.4	105.3	117.6	118.7	121.4
LL + I :	M-	0.0	0.0	0.0	0.0	-0.0	-0.0	-0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
LL + I :	Vmx	60.8	74.9	89.7	105.3	117.6	118.7	121.4
	M	2614.8	2402.0	1899.5	1079.9	278.5	198.6	0.0
Total :	M+	9347.3	8163.1	6179.3	3376.9	851.4	605.9	0.0
	V	47.7	109.7	204.0	259.1	299.1	302.6	311.2
Total :	M-	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total :	Vmx	99.4	151.7	205.4	259.1	299.1	302.6	311.2
	M	8905.6	7887.0	6040.1	3337.8	847.0	603.1	0.0

REACTIONS (kips), SERVICE I

Load Type	Left Support	Right Support
Self Wt.	68.7	68.7
Deck+Haunch	57.2	57.2
Diaphragm	0.8	0.8
DL-Prec.(DC)	18.0	18.0
DL-Prec.(DW)	0.0	0.0
DL-Comp.(DC)	93.7	93.7
DL-Comp.(DW)	347.1	347.1
Live	111.3	111.3
Pedestrian	0.0	0.0

Upward reactions are positive.
Live Load reactions are per lane with no distribution factor and no impact.
Non-composite load types are per beam.
Composite and Pedestrian load types are per total bridge width.

Bentley			Sheet #	DS-11
			Job #	49633
Program:	LEAP® CONSPAN® V8i (SELECTseries 2)	Great Lakes Client Licenses	Designed By	AKS
Version:	10.00.02.19	Copyright © Bentley Systems, Inc. 2011	Date	Apr/18/2011
		www.bentley.com Phone: 1-800-778-4277	Checked	DBT
File Name:	**CUY-90-1627-date120210-FINAL.csl**		Date	Apr/18/2011



BR01 - Precast Moment, Span 1, Beam 2, SERVICE I

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			Job #	49633
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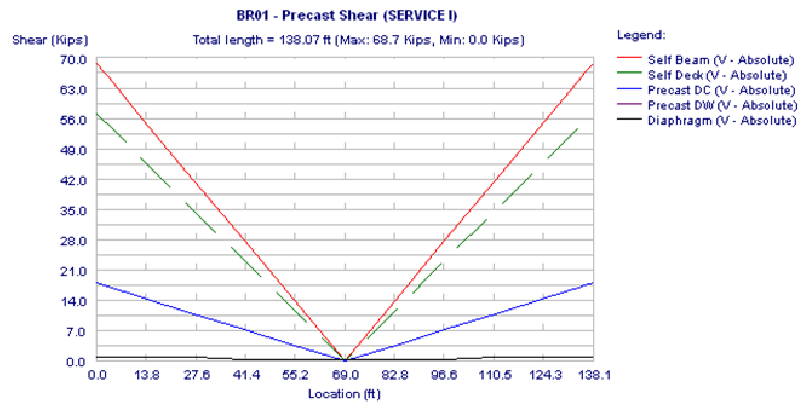
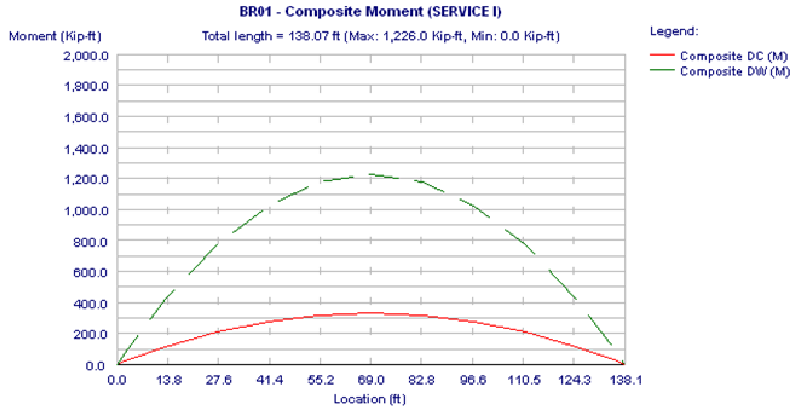
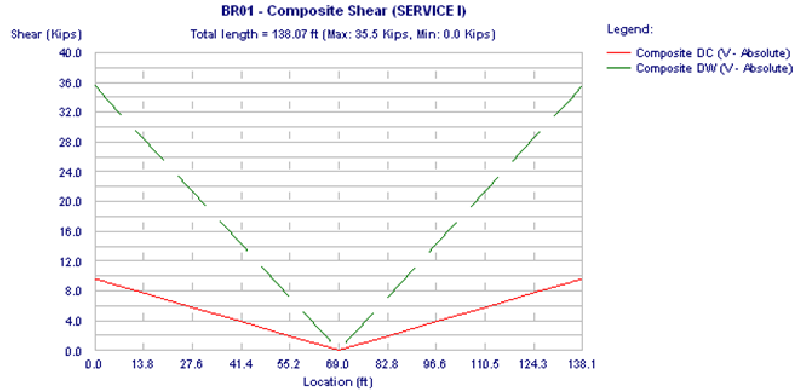
Version:	10.00.02.19	Copyright © Bentley Systems, Inc. 2011	Date	Apr/18/2011
		www.bentley.com Phone: 1-800-778-4277	Checked	DBT
File Name:	**CUY-90-1627-date120210-FINAL.csl**		Date	Apr/18/2011



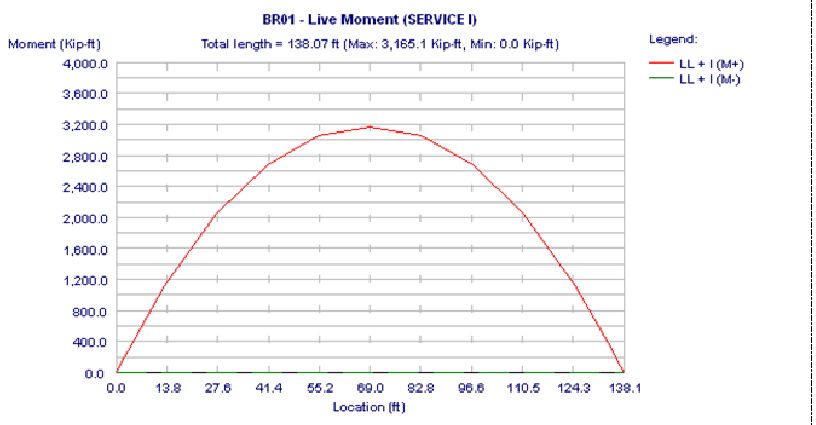
BR01 - Precast Shear, Span 1, Beam 2, SERVICE I



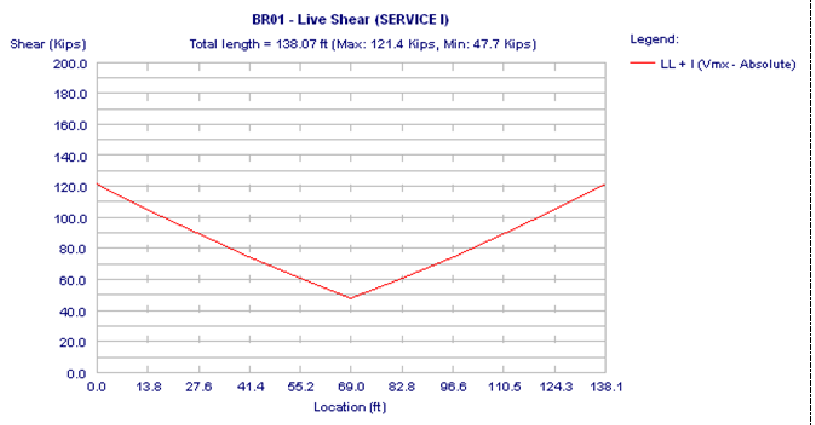
BR01 - Composite Moment, Span 1, Beam 2, SERVICE I



BR01 - Composite Shear, Span 1, Beam 2, SERVICE I



BR01 - Live Moment, Span 1, Beam 2, SERVICE I



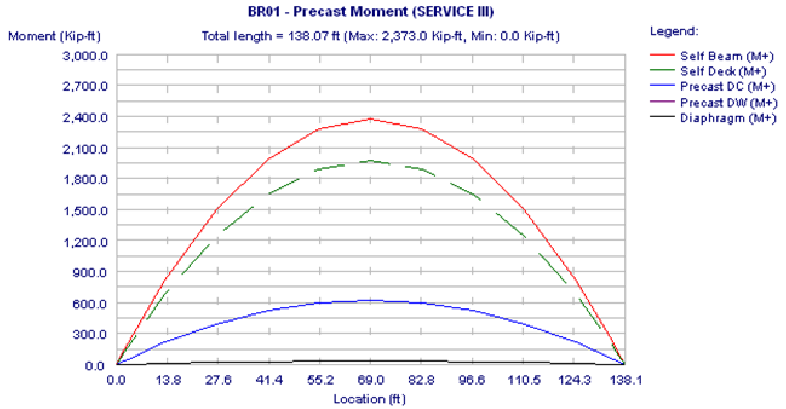
BR01 - Live Shear, Span 1, Beam 2, SERVICE I

SHEAR AND MOMENT ENVELOPE : Span : 1, Beam : 2, SERVICE III
Shears: kips, Moments: kft

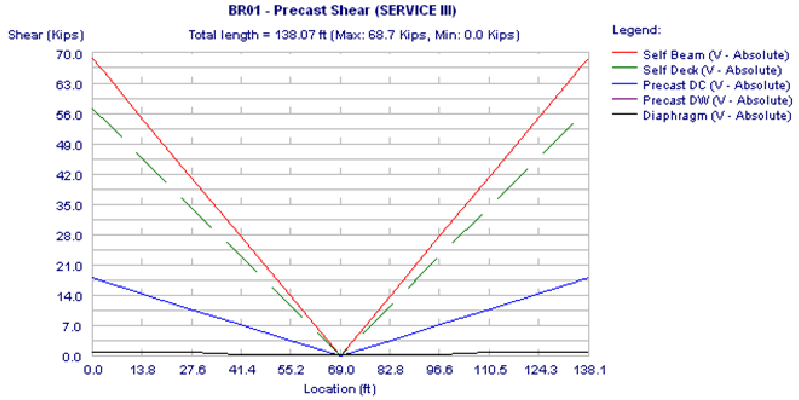
		Bearing	Trans	H/2	0.10L	0.20L	0.30L	0.40L	Midspan
Location,	ft	0.00	2.17	3.06	13.14	27.11	41.09	55.06	69.04
Self wt. :	M	0.0	146.5	205.9	817.3	1497.9	1984.1	2275.8	2373.0
(Max)	V	68.7	66.6	65.7	55.7	41.7	27.8	13.9	0.0
DL-Prec. :	M	0.0	38.3	53.8	213.7	391.7	518.8	595.1	620.5
DC(Max)	V	18.0	17.4	17.2	14.6	10.9	7.3	3.6	0.0
DL-Prec. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DW(Max)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Deck + :	M	0.0	121.8	171.2	679.7	1245.7	1649.9	1892.5	1973.3
Haunch (Max)	V	57.2	55.4	54.6	46.3	34.7	23.1	11.6	0.0
Diaphragm :	M	0.0	1.8	2.5	10.8	22.4	30.3	34.1	38.0
(Max)	V	0.8	0.8	0.8	0.8	0.8	0.3	0.3	0.3
DL-Comp :	M	0.0	20.4	28.7	114.1	209.0	276.9	317.6	331.1
DC(Max)	V	9.6	9.3	9.2	7.8	5.8	3.9	1.9	0.0
DL-Comp :	M	0.0	75.7	106.4	422.3	773.9	1025.1	1175.8	1226.0
DW(Max)	V	35.5	34.4	33.9	28.8	21.6	14.4	7.2	0.0

		Bearing	Trans	H/2	0.10L	0.20L	0.30L	0.40L	Midspan
LL + I :	M+	0.0	161.1	226.3	895.2	1630.9	2142.5	2445.1	2532.1
	V	97.1	95.0	94.1	84.2	70.7	26.3	7.3	24.7
LL + I :	M-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
LL + I :	Vmx	97.1	95.0	94.1	84.2	71.8	59.9	48.7	38.2
	M	0.0	158.9	222.8	863.9	1519.6	1921.6	2091.8	2056.7
Total :	M+	0.0	565.6	794.8	3153.1	5771.5	7627.5	8736.0	9094.1
	V	286.9	278.9	275.6	238.1	186.3	103.1	45.9	25.0
Total :	M-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total :	Vmx	286.9	278.9	275.6	238.1	187.4	136.7	87.2	38.4
	M	0.0	563.4	791.3	3121.8	5660.2	7406.6	8382.7	8618.7

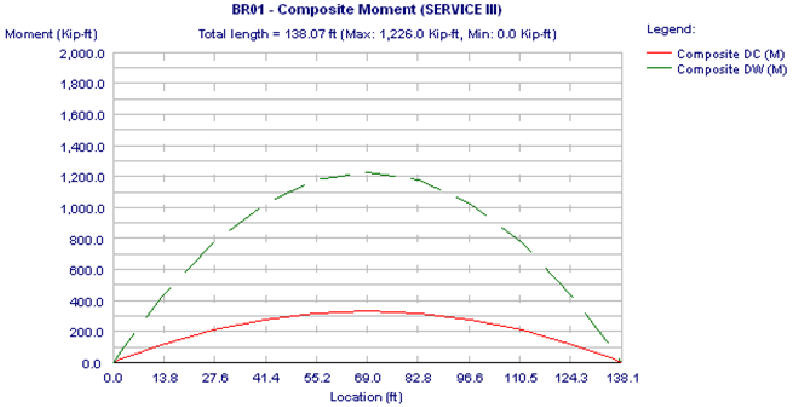
		0.60L	0.70L	0.80L	0.90L	H/2	Trans	Bearing
Location,	ft	83.01	96.98	110.96	124.93	135.01	135.91	138.07
Self wt. :	M	2275.8	1984.1	1497.9	817.3	205.9	146.5	0.0
(Max)	V	13.9	27.8	41.7	55.7	65.7	66.6	68.7
DL-Prec. :	M	595.1	518.8	391.7	213.7	53.8	38.3	0.0
DC(Max)	V	3.6	7.3	10.9	14.6	17.2	17.4	18.0
DL-Prec. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DW(Max)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Deck + :	M	1892.5	1649.9	1245.7	679.7	171.2	121.8	0.0
Haunch (Max)	V	11.6	23.1	34.7	46.3	54.6	55.4	57.2
Diaphragm :	M	34.1	30.3	22.4	10.8	2.5	1.8	-0.0
(Max)	V	0.3	0.3	0.8	0.8	0.8	0.8	0.8
DL-Comp :	M	317.6	276.9	209.0	114.1	28.7	20.4	-0.0
DC(Max)	V	1.9	3.9	5.8	7.8	9.2	9.3	9.6
DL-Comp :	M	1175.8	1025.1	773.9	422.3	106.4	75.7	0.0
DW(Max)	V	7.2	14.4	21.6	28.8	33.9	34.4	35.5
LL + I :	M+	2445.1	2142.5	1630.9	895.2	226.3	161.1	-0.0
	V	7.3	26.3	70.7	84.2	94.1	95.0	97.1
LL + I :	M-	0.0	0.0	0.0	0.0	-0.0	-0.0	-0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
LL + I :	Vmx	48.7	59.9	71.8	84.2	94.1	95.0	97.1
	M	2091.8	1921.6	1519.6	863.9	222.8	158.9	0.0
Total :	M+	8736.0	7627.5	5771.5	3153.1	794.8	565.6	0.0
	V	45.9	103.1	186.3	238.1	275.6	278.9	286.9
Total :	M-	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total :	Vmx	87.2	136.7	187.4	238.1	275.6	278.9	286.9
	M	8382.7	7406.6	5660.2	3121.8	791.3	563.4	0.0



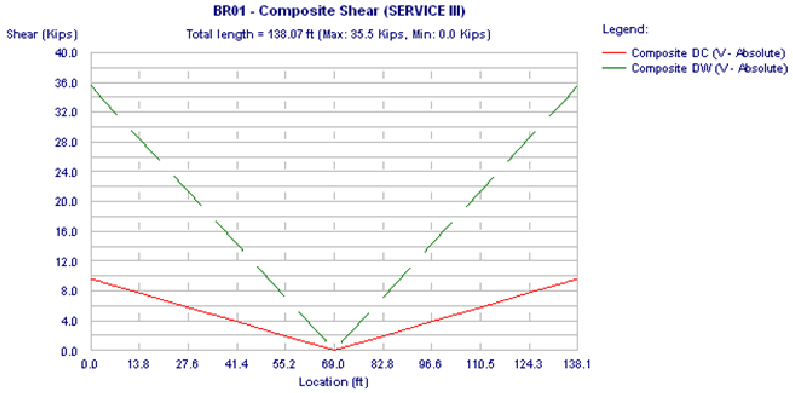
BR01 - Precast Moment, Span 1, Beam 2, SERVICE III



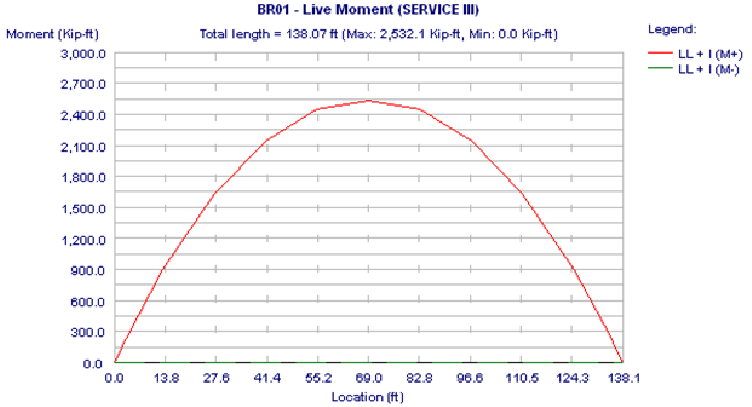
BR01 - Precast Shear, Span 1, Beam 2, SERVICE III



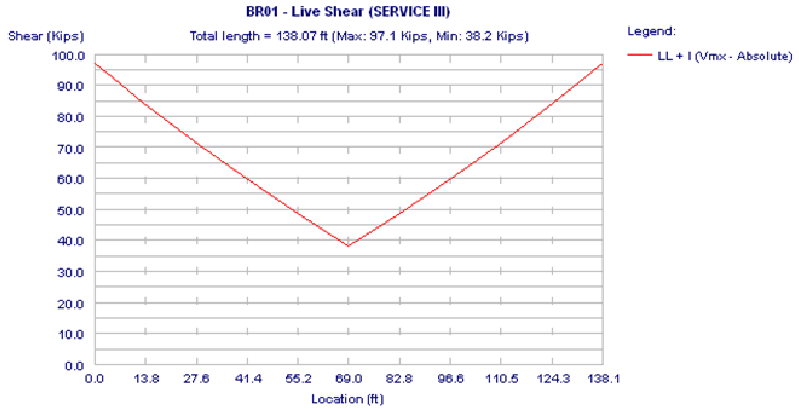
BR01 - Composite Moment, Span 1, Beam 2, SERVICE III



BR01 - Composite Shear, Span 1, Beam 2, SERVICE III



BR01 - Live Moment, Span 1, Beam 2, SERVICE III



BR01 - Live Shear, Span 1, Beam 2, SERVICE III

SHEAR AND MOMENT ENVELOPE : Span : 1, Beam : 2, STRENGTH I
Shears: kips, Moments: kft

		Bearing	Trans	H/2	0.10L	0.20L	0.30L	0.40L	Midspan
Location,	ft	0.00	2.17	3.06	13.14	27.11	41.09	55.06	69.04
Self wt. :	M	0.0	183.1	257.3	1021.6	1872.4	2480.1	2844.7	2966.2
(Max)	V	85.9	83.2	82.1	69.6	52.2	34.8	17.4	0.0
Self wt. :	M	0.0	131.9	185.3	735.6	1348.1	1785.7	2048.2	2135.7
(Min)	V	61.9	59.9	59.1	50.1	37.6	25.0	12.5	0.0
DL-Prec. :	M	0.0	47.9	67.3	267.2	489.6	648.6	743.9	775.7
DC(Max)	V	22.5	21.8	21.5	18.2	13.6	9.1	4.5	0.0
DL-Prec. :	M	0.0	34.5	48.5	192.4	352.5	467.0	535.6	558.5
DC(Min)	V	16.2	15.7	15.5	13.1	9.8	6.6	3.3	0.0
DL-Prec. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DW(Max)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DL-Prec. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DW(Min)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Deck + :	M	0.0	152.3	214.0	849.6	1557.1	2062.4	2365.6	2466.7
Haunch (Max)	V	71.5	69.2	68.3	57.9	43.4	28.9	14.5	0.0

		Bearing	Trans	H/2	0.10L	0.20L	0.30L	0.40L	Midspan
Deck + :	M	0.0	109.6	154.1	611.7	1121.1	1484.9	1703.2	1776.0
Haunch (Min)	V	51.5	49.8	49.2	41.7	31.2	20.8	10.4	0.0
Diaphragm :	M	0.0	2.2	3.2	13.5	28.0	37.9	42.7	47.5
(Max)	V	1.0	1.0	1.0	1.0	1.0	0.3	0.3	0.3
Diaphragm :	M	0.0	1.6	2.3	9.8	20.1	27.3	30.7	34.2
(Min)	V	0.7	0.7	0.7	0.7	0.7	0.2	0.2	0.2
DL-Comp :	M	0.0	25.6	35.9	142.6	261.3	346.1	397.0	413.9
DC(Max)	V	12.0	11.6	11.5	9.7	7.3	4.9	2.4	0.0
DL-Comp :	M	0.0	18.4	25.9	102.6	188.1	249.2	285.8	298.0
DC(Min)	V	8.6	8.4	8.3	7.0	5.2	3.5	1.7	0.0
DL-Comp :	M	0.0	113.5	159.5	633.4	1160.9	1537.6	1763.7	1839.0
DW(Max)	V	53.3	51.6	50.9	43.1	32.4	21.6	10.8	0.0
DL-Comp :	M	0.0	49.2	69.1	274.5	503.0	666.3	764.3	796.9
DW(Min)	V	23.1	22.4	22.1	18.7	14.0	9.3	4.7	0.0
LL + I :	M+	0.0	352.3	495.0	1958.2	3567.7	4686.6	5348.7	5538.9
	V	212.4	207.8	205.9	184.2	154.8	57.5	16.1	54.1
LL + I :	M-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
LL + I :	Vmx	212.4	207.8	205.9	184.3	157.1	131.1	106.5	83.5
	M	0.0	347.5	487.4	1889.8	3324.2	4203.4	4575.9	4499.0
Total :	M+	0.0	877.0	1232.2	4886.2	8936.9	11799.2	13506.2	14047.9
	V	458.6	446.3	441.2	383.7	304.7	157.1	66.0	54.5
Total :	M-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total :	Vmx	458.6	446.3	441.2	383.8	307.0	230.6	156.4	83.8
	M	0.0	872.1	1224.6	4817.7	8693.4	11316.1	12733.4	13008.0

		0.60L	0.70L	0.80L	0.90L	H/2	Trans	Bearing
Location,	ft	83.01	96.98	110.96	124.93	135.01	135.91	138.07
Self wt. :	M	2844.7	2480.1	1872.4	1021.6	257.3	183.1	0.0
(Max)	V	17.4	34.8	52.2	69.6	82.1	83.2	85.9
Self wt. :	M	2048.2	1785.7	1348.1	735.6	185.3	131.9	0.0
(Min)	V	12.5	25.0	37.6	50.1	59.1	59.9	61.9
DL-Prec. :	M	743.9	648.6	489.6	267.2	67.3	47.9	0.0
DC(Max)	V	4.5	9.1	13.6	18.2	21.5	21.8	22.5
DL-Prec. :	M	535.6	467.0	352.5	192.4	48.5	34.5	0.0
DC(Min)	V	3.3	6.6	9.8	13.1	15.5	15.7	16.2
DL-Prec. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DW(Max)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DL-Prec. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DW(Min)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Deck + :	M	2365.6	2062.4	1557.1	849.6	214.0	152.3	0.0
Haunch (Max)	V	14.5	28.9	43.4	57.9	68.3	69.2	71.5
Deck + :	M	1703.2	1484.9	1121.1	611.7	154.1	109.6	0.0
Haunch (Min)	V	10.4	20.8	31.2	41.7	49.2	49.8	51.5
Diaphragm :	M	42.7	37.9	28.0	13.5	3.2	2.2	-0.0
(Max)	V	0.3	0.3	1.0	1.0	1.0	1.0	1.0
Diaphragm :	M	30.7	27.3	20.1	9.8	2.3	1.6	-0.0
(Min)	V	0.2	0.2	0.7	0.7	0.7	0.7	0.7
DL-Comp :	M	397.0	346.1	261.3	142.6	35.9	25.6	-0.0

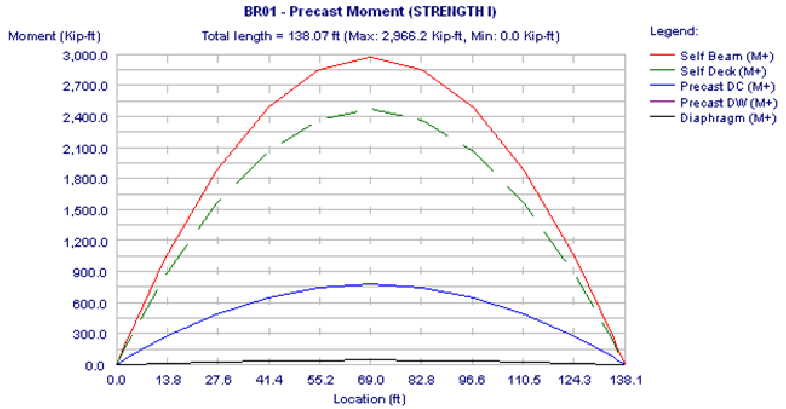
		0.60L	0.70L	0.80L	0.90L	H/2	Trans	Bearing
DC(Max)	V	2.4	4.9	7.3	9.7	11.5	11.6	12.0
DL-Comp :	M	285.8	249.2	188.1	102.6	25.9	18.4	-0.0
DC(Min)	V	1.7	3.5	5.2	7.0	8.3	8.4	8.6
DL-Comp :	M	1763.7	1537.6	1160.9	633.4	159.5	113.5	0.0
DW(Max)	V	10.8	21.6	32.4	43.1	50.9	51.6	53.3
DL-Comp :	M	764.3	666.3	503.0	274.5	69.1	49.2	0.0
DW(Min)	V	4.7	9.3	14.0	18.7	22.1	22.4	23.1
LL + I :	M+	5348.7	4686.6	3567.7	1958.2	495.0	352.3	-0.0
	V	16.1	57.5	154.8	184.2	205.9	207.8	212.4
LL + I :	M-	0.0	0.0	0.0	0.0	-0.0	-0.0	-0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
LL + I :	Vmx	106.5	131.1	157.1	184.3	205.9	207.8	212.4
	M	4575.9	4203.4	3324.2	1889.8	487.4	347.5	0.0
Total :	M+	13506.2	11799.2	8936.9	4886.2	1232.2	877.0	0.0
	V	66.0	157.1	304.7	383.7	441.2	446.3	458.6
Total :	M-	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total :	Vmx	156.4	230.6	307.0	383.8	441.2	446.3	458.6
	M	12733.4	11316.1	8693.4	4817.7	1224.6	872.1	0.0

REACTIONS (kips), STRENGTH I

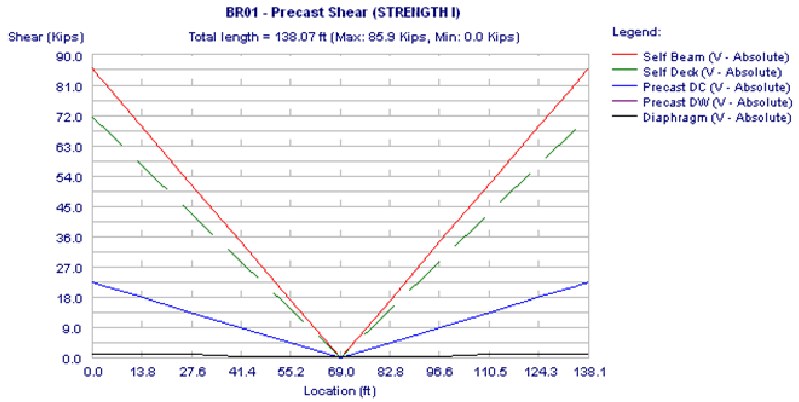
Load Type	Left Support	Right Support
Self Wt.	85.9	85.9
Deck+Haunch	71.5	71.5
Diaphragm	1.0	1.0
DL-Prec.(DC)	22.5	22.5
DL-Prec.(DW)	0.0	0.0
DL-Comp.(DC)	117.2	117.2
DL-Comp.(DW)	520.6	520.6
Live	194.8	194.8
Pedestrian	0.0	0.0

Upward reactions are positive.
Live Load reactions are per lane with no distribution factor and no impact.
Non-composite load types are per beam.
Composite and Pedestrian load types are per total bridge width.

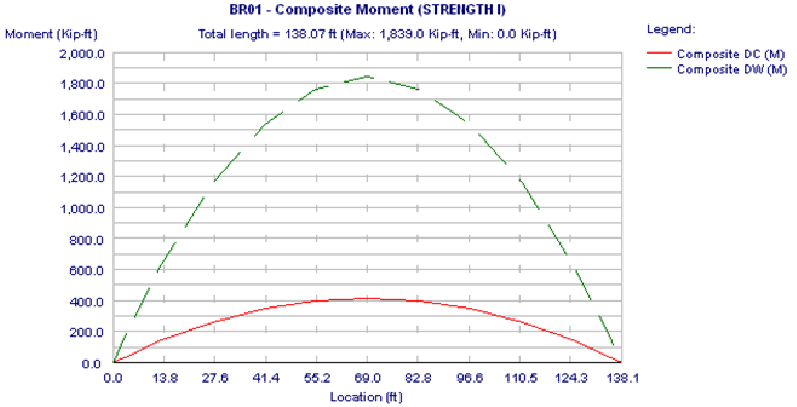
Bentley			Sheet #	DS-26
			Job #	49633
Program:	LEAP® CONSPAN® V8i (SELECTseries 2)	Great Lakes Client Licenses	Designed By	AKS
Version:	10.00.02.19	Copyright © Bentley Systems, Inc. 2011	Date	Apr/18/2011
		www.bentley.com Phone: 1-800-778-4277	Checked	DBT
File Name:	CUY-90-1627-date120210-FINAL.csl		Date	Apr/18/2011



BR01 - Precast Moment, Span 1, Beam 2, STRENGTH I



BR01 - Precast Shear, Span 1, Beam 2, STRENGTH I



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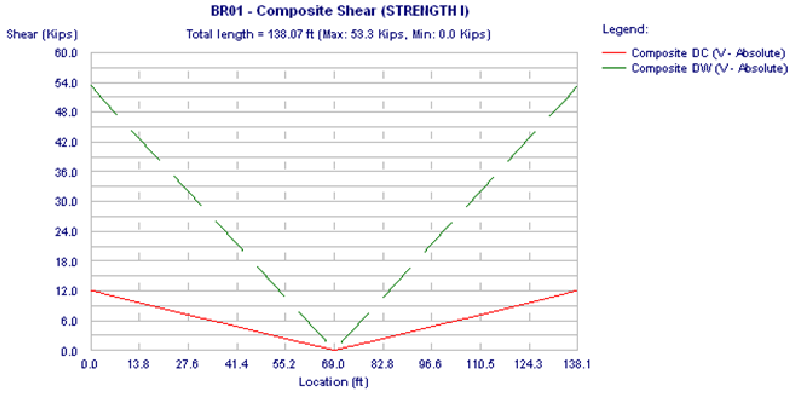

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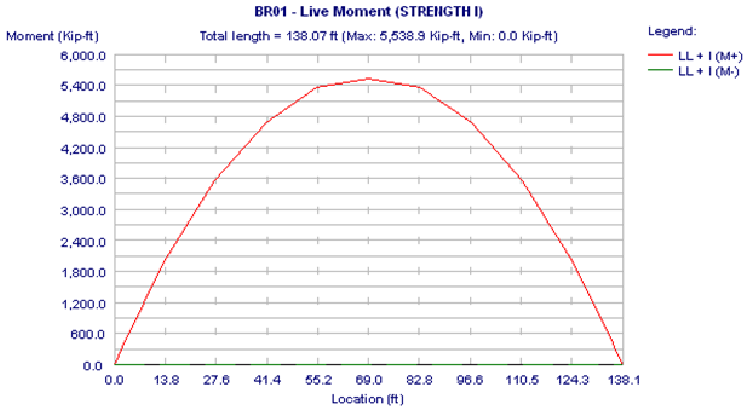

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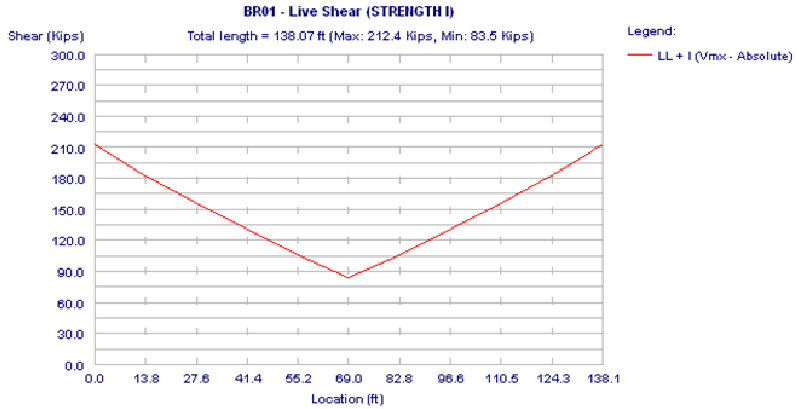


BR01 - Live Shear, Span 1, Beam 2, STRENGTH I

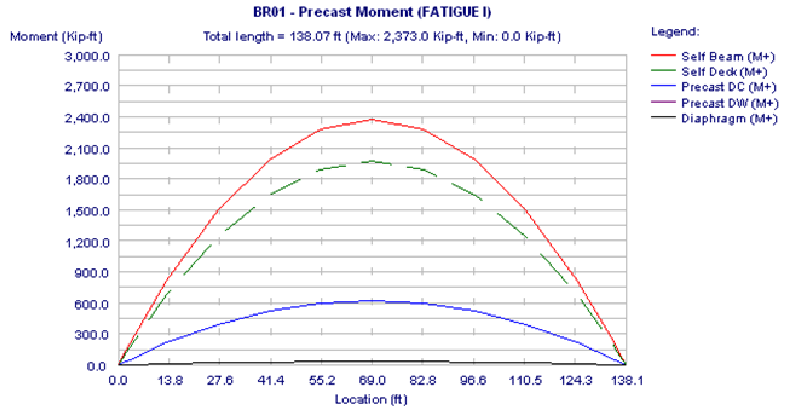
SHEAR AND MOMENT ENVELOPE : Span : 1, Beam : 2, FATIGUE I
Shears: kips, Moments: kft

		Bearing	Trans	H/2	0.10L	0.20L	0.30L	0.40L	Midspan
Location,	ft	0.00	2.17	3.06	13.14	27.11	41.09	55.06	69.04
Self wt. :	M	0.0	146.5	205.9	817.3	1497.9	1984.1	2275.8	2373.0
(Max)	V	68.7	66.6	65.7	55.7	41.7	27.8	13.9	0.0
Self wt. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
(Min)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DL-Prec. :	M	0.0	38.3	53.8	213.7	391.7	518.8	595.1	620.5
DC(Max)	V	18.0	17.4	17.2	14.6	10.9	7.3	3.6	0.0
DL-Prec. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DC(Min)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DL-Prec. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DW(Max)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DL-Prec. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DW(Min)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Deck + :	M	0.0	121.8	171.2	679.7	1245.7	1649.9	1892.5	1973.3
Haunch (Max)	V	57.2	55.4	54.6	46.3	34.7	23.1	11.6	0.0

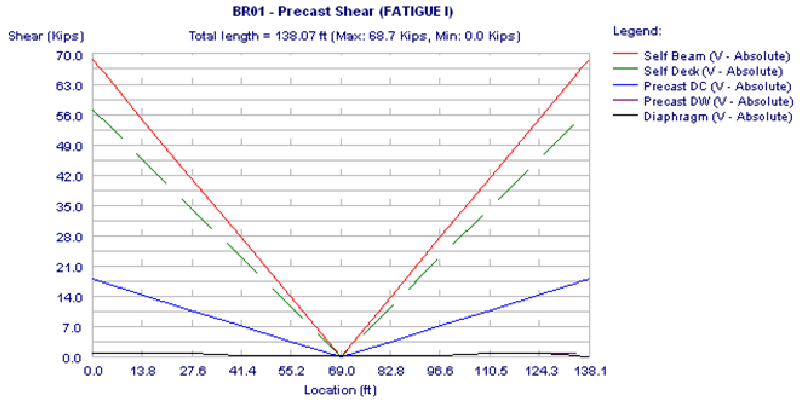
		Bearing	Trans	H/2	0.10L	0.20L	0.30L	0.40L	Midspan
Deck + :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Haunch (Min)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Diaphragm :	M	0.0	1.8	2.5	10.8	22.4	30.3	34.1	38.0
(Max)	V	0.8	0.8	0.8	0.8	0.8	0.3	0.3	0.3
Diaphragm :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
(Min)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DL-Comp :	M	0.0	20.4	28.7	114.1	209.0	276.9	317.6	331.1
DC(Max)	V	9.6	9.3	9.2	7.8	5.8	3.9	1.9	0.0
DL-Comp :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DC(Min)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DL-Comp :	M	0.0	75.7	106.4	422.3	773.9	1025.1	1175.8	1226.0
DW(Max)	V	35.5	34.4	33.9	28.8	21.6	14.4	7.2	0.0
DL-Comp :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DW(Min)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
LL + I :	M+	0.0	90.7	127.3	499.4	896.4	1174.9	1326.6	1339.9
	V	66.0	64.7	64.2	58.4	50.8	38.3	30.7	23.1
LL + I :	M-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
LL + I :	Vmx	66.0	64.7	64.2	58.4	50.8	43.2	35.6	28.0
	M	0.0	90.7	127.3	499.4	896.4	1155.1	1275.4	1257.4
Total :	M+	0.0	495.3	695.9	2757.3	5037.0	6659.9	7617.4	7901.9
	V	255.8	248.6	245.7	212.2	166.4	115.1	69.2	23.4
Total :	M-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total :	Vmx	255.8	248.6	245.7	212.2	166.4	120.0	74.1	28.2
	M	0.0	495.3	695.9	2757.3	5037.0	6640.2	7566.3	7819.5

		0.60L	0.70L	0.80L	0.90L	H/2	Trans	Bearing
Location,	ft	83.01	96.98	110.96	124.93	135.01	135.91	138.07
Self wt. :	M	2275.8	1984.1	1497.9	817.3	205.9	146.5	0.0
(Max)	V	13.9	27.8	41.7	55.7	65.7	66.6	68.7
Self wt. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0
(Min)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DL-Prec. :	M	595.1	518.8	391.7	213.7	53.8	38.3	0.0
DC(Max)	V	3.6	7.3	10.9	14.6	17.2	17.4	18.0
DL-Prec. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DC(Min)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DL-Prec. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DW(Max)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DL-Prec. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DW(Min)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Deck + :	M	1892.5	1649.9	1245.7	679.7	171.2	121.8	0.0
Haunch (Max)	V	11.6	23.1	34.7	46.3	54.6	55.4	57.2
Deck + :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Haunch (Min)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Diaphragm :	M	34.1	30.3	22.4	10.8	2.5	1.8	-0.0
(Max)	V	0.3	0.3	0.8	0.8	0.8	0.8	0.8
Diaphragm :	M	0.0	0.0	0.0	0.0	0.0	0.0	-0.0
(Min)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DL-Comp :	M	317.6	276.9	209.0	114.1	28.7	20.4	-0.0

		0.60L	0.70L	0.80L	0.90L	H/2	Trans	Bearing
DC(Max)	V	1.9	3.9	5.8	7.8	9.2	9.3	9.6
DL-Comp :	M	0.0	0.0	0.0	0.0	0.0	0.0	-0.0
DC(Min)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DL-Comp :	M	1175.8	1025.1	773.9	422.3	106.4	75.7	0.0
DW(Max)	V	7.2	14.4	21.6	28.8	33.9	34.4	35.5
DL-Comp :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DW(Min)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
LL + I :	M+	1326.6	1174.9	896.4	499.4	127.3	90.7	0.0
	V	30.7	38.3	50.8	58.4	64.2	64.7	66.0
LL + I :	M-	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
LL + I :	Vmx	35.6	43.2	50.8	58.4	64.2	64.7	66.0
	M	1275.4	1155.1	896.4	499.4	127.3	90.7	0.0
Total :	M+	7617.4	6659.9	5037.0	2757.3	695.9	495.3	0.0
	V	69.2	115.1	166.4	212.2	245.7	248.6	255.8
Total :	M-	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total :	Vmx	74.1	120.0	166.4	212.2	245.7	248.6	255.8
	M	7566.3	6640.2	5037.0	2757.3	695.9	495.3	0.0

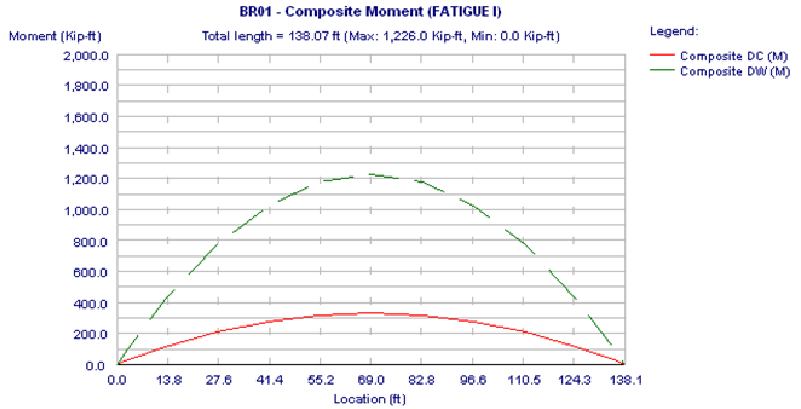


BR01 - Precast Moment, Span 1, Beam 2, FATIGUE I



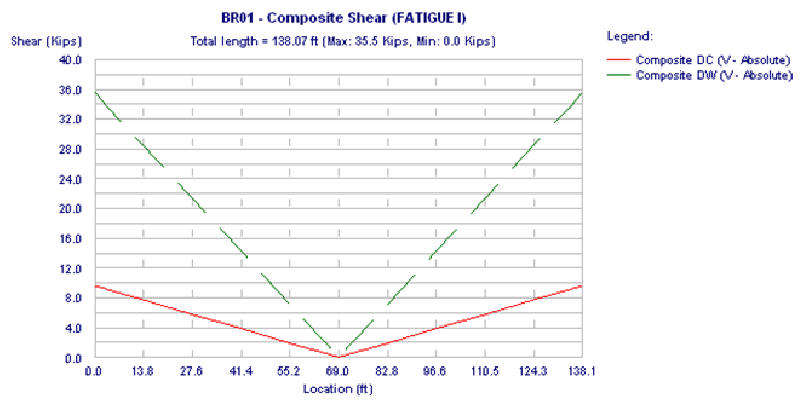
BR01 - Precast Shear, Span 1, Beam 2, FATIGUE I

Bentley			Sheet #	DS-35
			Job #	49633
Program:	LEAP® CONSPAN® V8i (SELECTseries 2)	Great Lakes Client Licenses	Designed By	AKS
Version:	10.00.02.19	Copyright © Bentley Systems, Inc. 2011	Date	Apr/18/2011
		www.bentley.com Phone: 1-800-778-4277	Checked	DBT
File Name:	**CUY-90-1627-date120210-FINAL.csl**		Date	Apr/18/2011



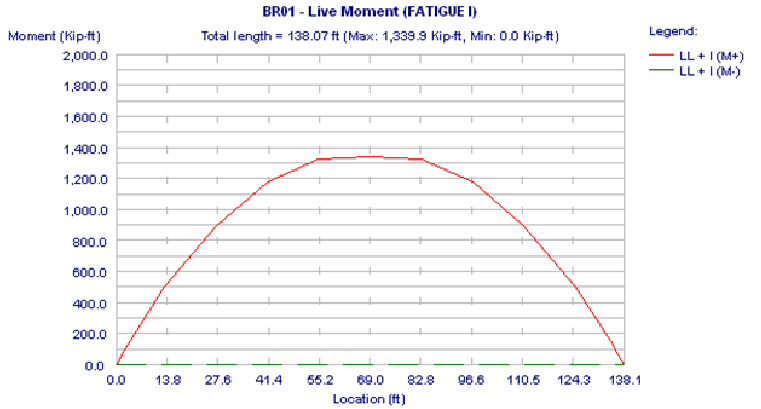
BR01 - Composite Moment, Span 1, Beam 2, FATIGUE I

Bentley			Sheet #	DS-36
			Job #	49633
Program:	LEAP® CONSPAN® V8i (SELECTseries 2)	Great Lakes Client Licenses	Designed By	AKS
Version:	10.00.02.19	Copyright © Bentley Systems, Inc. 2011	Date	Apr/18/2011
		www.bentley.com Phone: 1-800-778-4277	Checked	DBT
File Name:	**CUY-90-1627-date120210-FINAL.csl**		Date	Apr/18/2011



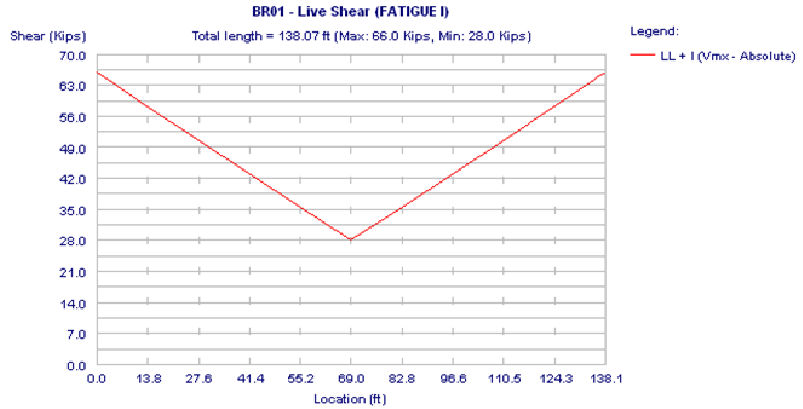
BR01 - Composite Shear, Span 1, Beam 2, FATIGUE I

Bentley			Sheet #	DS-37
			Job #	49633
Program:	LEAP® CONSPAN® V8i (SELECTseries 2)	Great Lakes Client Licenses	Designed By	AKS
Version:	10.00.02.19	Copyright © Bentley Systems, Inc. 2011	Date	Apr/18/2011
		www.bentley.com Phone: 1-800-778-4277	Checked	DBT
File Name:	**CUY-90-1627-date120210-FINAL.csl**		Date	Apr/18/2011



BR01 - Live Moment, Span 1, Beam 2, FATIGUE I

Bentley			Sheet #	DS-38
			Job #	49633
Program:	LEAP® CONSPAN® V8i (SELECTseries 2)	Great Lakes Client Licenses	Designed By	AKS
Version:	10.00.02.19	Copyright © Bentley Systems, Inc. 2011	Date	Apr/18/2011
		www.bentley.com Phone: 1-800-778-4277	Checked	DBT
File Name:	**CUY-90-1627-date120210-FINAL.csl**		Date	Apr/18/2011



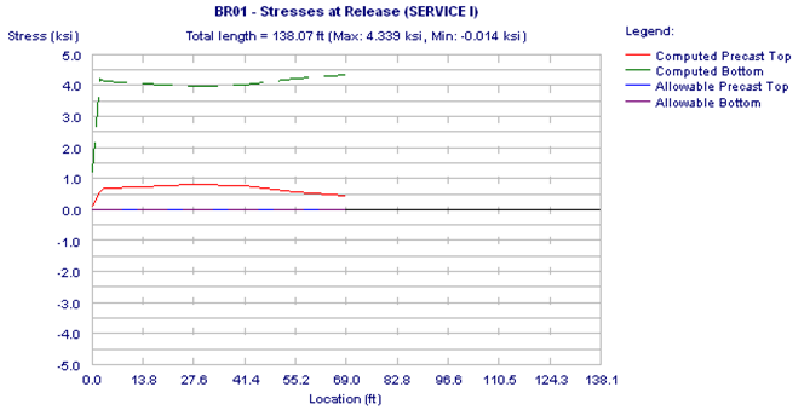
BR01 - Live Shear, Span 1, Beam 2, FATIGUE I

POSITIVE ENVELOPE STRESSES

Span : 1, Beam : 2, SERVICE I

RELEASE STRESSES, (ksi) (LOSS = 10.71 %)

	Trans	0.10L /0.90L	0.20L /0.80L	0.30L /0.70L	0.40L /0.60L	Midspan	Depress.
Location, ft	3.00	13.97	27.95	41.92	55.90	69.87	62.88
Beam-Self							
Precast-top	0.149	0.638	1.133	1.488	1.700	1.771	1.753
Bottom	-0.151	-0.645	-1.147	-1.506	-1.721	-1.793	-1.775
Prestress							
Precast-top	0.410	0.092	-0.313	-0.718	-1.123	-1.326	-1.326
Bottom	4.375	4.697	5.107	5.517	5.927	6.132	6.132
Total							
Precast-top	0.559	0.730	0.820	0.770	0.577	0.446	0.428
Bottom	4.224	4.051	3.959	4.011	4.206	4.339	4.357



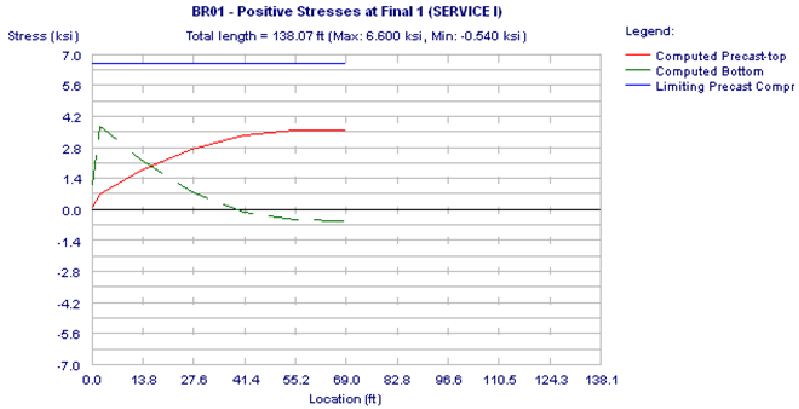
BR01 - Stresses at Release, Span 1, Beam 2, SERVICE I

SERVICE I

POSITIVE ENVELOPE STRESSES, (ksi) (LOSS = 15.70 %)

	Bearing	Trans	H/2	0.10L /0.90L	0.20L /0.80L	0.30L /0.70L	0.40L /0.60L	Midspan
Location, ft	0.00	2.17	3.06	13.14	27.11	41.09	55.06	69.04
Prestress								
Precast-top	0.124	0.387	0.363	0.087	-0.296	-0.678	-1.060	-1.251
Bottom	1.133	4.130	4.155	4.434	4.821	5.208	5.595	5.789
Self wt.								
Precast-top	0.000	0.107	0.150	0.596	1.091	1.446	1.658	1.729
Bottom	-0.000	-0.108	-0.152	-0.603	-1.105	-1.463	-1.678	-1.750
DL-Prec (DC)								
Precast-top	0.000	0.028	0.039	0.156	0.285	0.378	0.434	0.452
Bottom	-0.000	-0.028	-0.040	-0.158	-0.289	-0.383	-0.439	-0.458

		Bearing	Trans	H/2	0.10L /0.90L	0.20L /0.80L	0.30L /0.70L	0.40L /0.60L	Midspan
DL-Prec (DW)									
Precast-top		-0.000	-0.000	-0.000	-0.000	-0.000	-0.000	-0.000	-0.000
Bottom		-0.000	-0.000	-0.000	-0.000	-0.000	-0.000	-0.000	-0.000
Diaphragm									
Precast-top		0.000	0.001	0.002	0.008	0.016	0.022	0.025	0.028
Bottom		-0.000	-0.001	-0.002	-0.008	-0.016	-0.022	-0.025	-0.028
Deck + Haunch									
Precast-top		0.000	0.089	0.125	0.495	0.908	1.202	1.379	1.438
Bottom		-0.000	-0.090	-0.126	-0.501	-0.919	-1.217	-1.396	-1.455
DL-Comp (DC)									
Precast-top		0.000	0.005	0.007	0.028	0.051	0.068	0.078	0.081
Bottom		-0.000	-0.011	-0.016	-0.064	-0.117	-0.155	-0.177	-0.185
DL-Comp (DW)									
Precast-top		0.000	0.019	0.026	0.103	0.189	0.251	0.288	0.300
Bottom		-0.000	-0.042	-0.059	-0.236	-0.432	-0.573	-0.657	-0.685
LL+I(+)									
Precast-top		0.000	0.049	0.069	0.274	0.499	0.655	0.748	0.775
Bottom		-0.000	-0.112	-0.158	-0.625	-1.139	-1.496	-1.707	-1.768
Final 1 (P/S + DL	+ LL)								
Precast-top		0.124	0.685	0.781	1.746	2.745	3.344	3.549	3.551
Bottom		1.133	3.737	3.602	2.240	0.805	-0.100	-0.484	-0.540
Final 2 (P/S + DL)									
Precast-top		0.124	0.635	0.711	1.472	2.246	2.689	2.801	2.776
Bottom		1.133	3.849	3.760	2.865	1.943	1.396	1.223	1.228



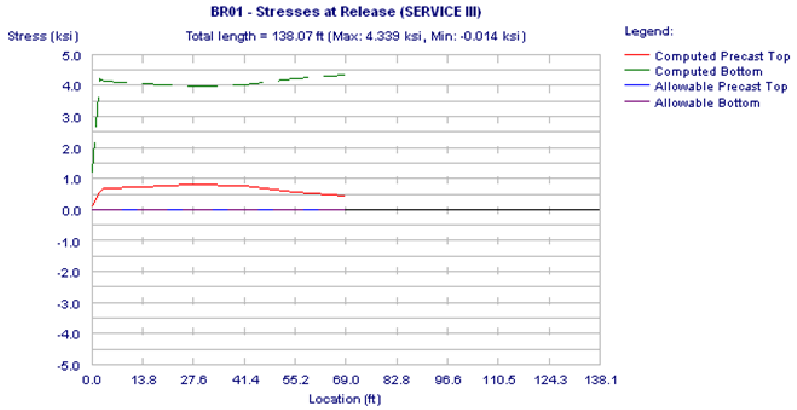
BR01 - Positive Stresses at Final 1, Span 1, Beam 2, SERVICE I

Span : 1, Beam : 2, SERVICE III

RELEASE STRESSES, (ksi) (LOSS = 10.71 %)

	Trans	0.10L /0.90L	0.20L /0.80L	0.30L /0.70L	0.40L /0.60L	Midspan	Depress.
Location, ft	3.00	13.97	27.95	41.92	55.90	69.87	62.88
Beam-Self							
Precast-top	0.149	0.638	1.133	1.488	1.700	1.771	1.753
Bottom	-0.151	-0.645	-1.147	-1.506	-1.721	-1.793	-1.775
Prestress							
Precast-top	0.410	0.092	-0.313	-0.718	-1.123	-1.326	-1.326
Bottom	4.375	4.697	5.107	5.517	5.927	6.132	6.132
Total							
Precast-top	0.559	0.730	0.820	0.770	0.577	0.446	0.428

		Trans	0.10L /0.90L	0.20L /0.80L	0.30L /0.70L	0.40L /0.60L	Midspan	Depress.
Bottom		4.224	4.051	3.959	4.011	4.206	4.339	4.357
As_top, in2		0.000	0.000	0.000	0.000	0.000	0.000	0.000
Ast_prvd, in2		0.000	0.000	0.000	0.000	0.000	0.000	0.000



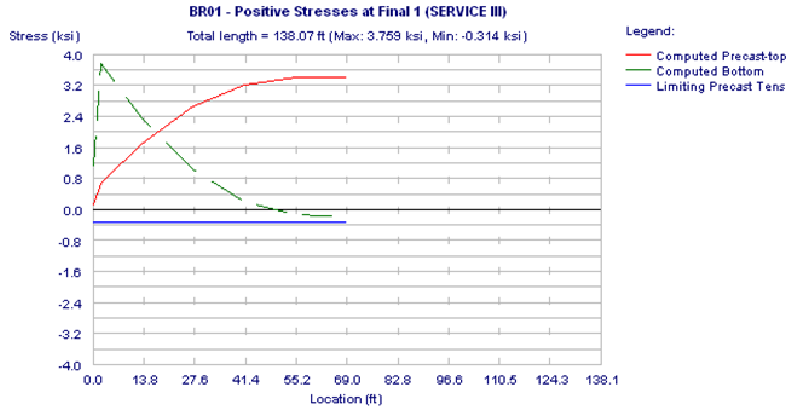
BR01 - Stresses at Release, Span 1, Beam 2, SERVICE III

SERVICE III

POSITIVE ENVELOPE STRESSES, (ksi) (LOSS = 15.70 %)

		Bearing	Trans	H/2	0.10L /0.90L	0.20L /0.80L	0.30L /0.70L	0.40L /0.60L	Midspan
Location, ft		0.00	2.17	3.06	13.14	27.11	41.09	55.06	69.04
Prestress									
Precast-top		0.124	0.387	0.363	0.087	-0.296	-0.678	-1.060	-1.251

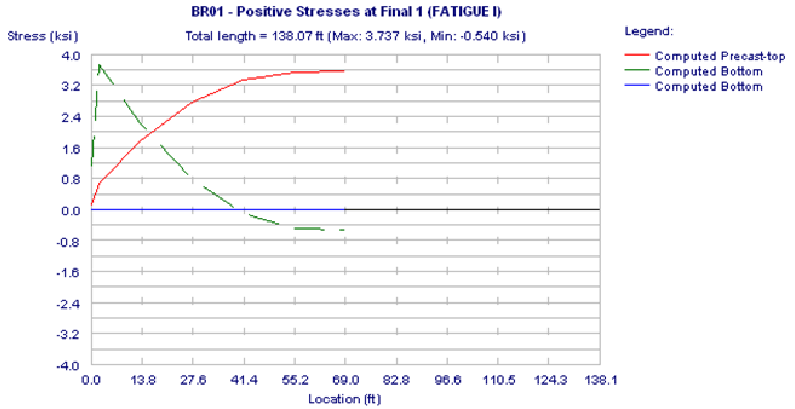
		Bearing	Trans	H/2	0.10L /0.90L	0.20L /0.80L	0.30L /0.70L	0.40L /0.60L	Midspan
Bottom		1.133	4.130	4.155	4.434	4.821	5.208	5.595	5.789
Self wt.									
Precast-top		0.000	0.107	0.150	0.596	1.091	1.446	1.658	1.729
Bottom		-0.000	-0.108	-0.152	-0.603	-1.105	-1.463	-1.678	-1.750
DL-Prec (DC)									
Precast-top		0.000	0.028	0.039	0.156	0.285	0.378	0.434	0.452
Bottom		-0.000	-0.028	-0.040	-0.158	-0.289	-0.383	-0.439	-0.458
DL-Prec (DW)									
Precast-top		-0.000	-0.000	-0.000	-0.000	-0.000	-0.000	-0.000	-0.000
Bottom		-0.000	-0.000	-0.000	-0.000	-0.000	-0.000	-0.000	-0.000
Diaphragm									
Precast-top		0.000	0.001	0.002	0.008	0.016	0.022	0.025	0.028
Bottom		-0.000	-0.001	-0.002	-0.008	-0.016	-0.022	-0.025	-0.028
Deck + Haunch									
Precast-top		0.000	0.089	0.125	0.495	0.908	1.202	1.379	1.438
Bottom		-0.000	-0.090	-0.126	-0.501	-0.919	-1.217	-1.396	-1.455
DL-Comp (DC)									
Precast-top		0.000	0.005	0.007	0.028	0.051	0.068	0.078	0.081
Bottom		-0.000	-0.011	-0.016	-0.064	-0.117	-0.155	-0.177	-0.185
DL-Comp (DW)									
Precast-top		0.000	0.019	0.026	0.103	0.189	0.251	0.288	0.300
Bottom		-0.000	-0.042	-0.059	-0.236	-0.432	-0.573	-0.657	-0.685
LL+I(+)									
Precast-top		0.000	0.039	0.055	0.219	0.399	0.524	0.598	0.620
Bottom		-0.000	-0.090	-0.126	-0.500	-0.911	-1.197	-1.366	-1.414
Final 1 (P/S + DL	+ LL)								
Precast-top		0.124	0.675	0.767	1.692	2.645	3.213	3.399	3.396
Bottom		1.133	3.759	3.634	2.365	1.033	0.199	-0.143	-0.186



BR01 - Positive Stresses at Final 1, Span 1, Beam 2, SERVICE III

Span : 1, Beam : 2, FATIGUE I
POSITIVE ENVELOPE STRESSES, (ksi)

		Bearing	Trans	H/2	0.10L /0.90L	0.20L /0.80L	0.30L /0.70L	0.40L /0.60L	Midspan
Location, ft		0.00	2.17	3.06	13.14	27.11	41.09	55.06	69.04
F_LL+I(+)									
Precast-top		-0.000	0.022	0.031	0.122	0.219	0.288	0.325	0.328
Bottom		-0.000	-0.051	-0.071	-0.279	-0.501	-0.656	-0.741	-0.748
Final 3 (50% P/S	+ 50%	DL + F_LL)							
Precast-top		0.062	0.340	0.387	0.858	1.342	1.632	1.725	1.716
Bottom		0.566	1.874	1.809	1.154	0.471	0.042	-0.129	-0.134



BR01 - Positive Stresses at Final 1, Span 1, Beam 2, FATIGUE I

VERTICAL/HORIZONTAL SHEAR

VERTICAL SHEAR (Art. 5.8) - Span : 1, Beam : 2, STRENGTH I
Using General Beta Theta Equation procedure - Art.5.8.3.4.2

Location(ft) / Mcor (kft)	Vu (kips) / a (in)	bv (in) / dv (in)	de (in) / fpo (ksi)	Aps (in2) / vu/fc	Vp (kips) / Vc-com (kips)	eps_x / Beta	Theta / Max.spc. (in)	Vs-reqd (kips) / min.Av/s (in2/ft)	Av/s (in2/ft) / pVn/Vu	Av-prvd (in2/ft) / Aps* (in2)	Al_reqd (in2)
Bearing :	0.83										
458.6	8.00	67.69	1.847	10.4	6.00e-3	50.0	450.1	1.603	1.600	0.00	
0.0	1.48	66.94	52.6	0.085	49.0	0.87	24.00	0.168	0.999*	1.656	
Transfer :	3.00										
446.3	8.00	67.69	9.331	37.5	-0.19e-3	28.3	152.4	0.253	0.800	0.00	
872.1	5.27	65.05	189.0	0.080	306.0	5.61	24.00	0.168	1.666	5.951	
Critical :	6.22										
427.9	8.00	67.69	9.331	37.5	-0.20e-3	28.3	132.1	0.220	0.229	0.00	
2052.3	6.16	64.61	189.0	0.077	305.9	5.65	24.00	0.168	1.011	7.120	
0.1L :	13.97										
383.8	8.00	67.69	9.331	37.5	-0.10e-3	28.6	111.4	0.191	0.229	0.00	
4817.8	8.17	63.60	189.0	0.069	277.5	5.20	24.00	0.168	1.051	9.331	
0.2L :	27.95										
307.0	8.00	62.75	11.284	37.5	-0.02e-3	28.9	64.5	0.168	0.229	0.00	
8693.4	8.21	58.65	189.0	0.059	239.1	4.86	24.00	0.168	1.166	11.284	
0.3L :	41.92										
230.6	8.00	62.72	12.586	37.5	0.37e-3	30.3	33.6	0.168	0.229	0.00	
11316.1	8.22	58.61	189.0	0.042	185.2	3.77	24.00	0.168	1.316	12.586	
0.4L :	55.90										
156.4	8.00	65.66	12.586	37.5	0.62e-3	31.2	0.0	0.168	0.229	0.00	
12733.4	8.23	61.54	189.0	0.025	168.9	3.27	24.00	0.168	1.856	12.586	
0.5L :	69.87										
83.8	8.00	67.12	12.586	0.0	0.51e-3	30.8	0.0	0.168	0.229	0.00	
13008.0	8.23	63.01	189.0	0.017	183.6	3.47	24.00	0.168	3.269	12.586	
0.6L :	83.84										
156.4	8.00	65.66	12.586	37.5	0.62e-3	31.2	0.0	0.168	0.229	0.00	
12733.4	8.23	61.54	189.0	0.025	168.9	3.27	24.00	0.168	1.856	12.586	
0.7L :	97.82										

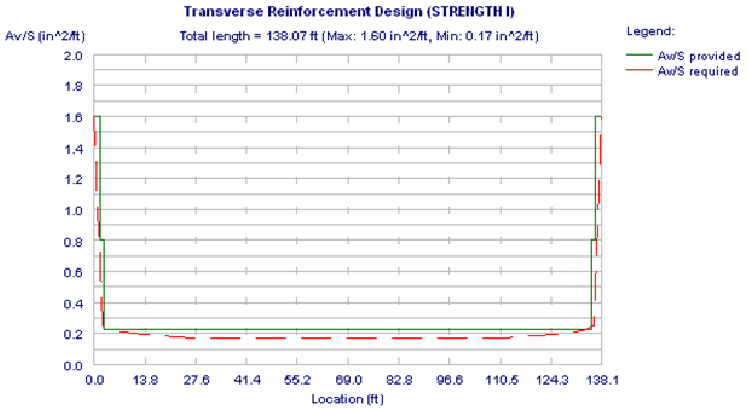
SAY OK

Location(ft) / Mcor (kft)	Vu (kips) / a (in)	bv (in) / dv (in)	de (in) / fpo (ksi)	Aps (in2) / vu/fc	Vp (kips) / Vc-com (kips)	eps_x / Beta	Theta / Max.spc. (in)	Vs-reqd (kips) / min.Av/s (in2/ft)	Av/s (in2/ft) / pVn/Vu	Av-prvd (in2/ft) / Aps* (in2)	Al_reqd (in2)
230.6	8.00	62.72	12.586	37.5	0.37e-3	30.3	33.6	0.168	0.229	0.00	
11316.1	8.22	58.61	189.0	0.042	185.2	3.77	24.00	0.168	1.316	12.586	
0.8L :	111.79										
307.0	8.00	62.75	11.284	37.5	-0.02e-3	28.9	64.5	0.168	0.229	0.00	
8693.4	8.21	58.65	189.0	0.059	239.1	4.86	24.00	0.168	1.166	11.284	
0.9L :	125.77										
383.8	8.00	67.69	9.331	37.5	-0.10e-3	28.6	111.4	0.191	0.229	0.00	
4817.7	8.17	63.60	189.0	0.069	277.5	5.20	24.00	0.168	1.051	9.331	
Critical :	133.52										
427.9	8.00	67.69	9.331	37.5	-0.20e-3	28.3	132.1	0.220	0.229	0.00	
2052.3	6.16	64.61	189.0	0.077	305.9	5.65	24.00	0.168	1.011	7.120	
Transfer :	136.74										
446.3	8.00	67.69	9.331	37.5	-0.19e-3	28.3	152.4	0.253	0.800	0.00	
872.1	5.27	65.05	189.0	0.080	306.0	5.61	24.00	0.168	1.666	5.951	
Bearing :	138.90										
458.6	8.00	67.69	1.847	10.4	6.00e-3	50.0	450.1	1.603	1.600	0.00	
0.0	1.48	66.94	52.6	0.085	49.0	0.87	24.00	0.168	0.999*	1.656	

SAY OK

ANCHORAGE ZONE REINFORCEMENT (Art. 5.10.10)
Span : 1, Beam : 2

Fpi (kips)	fs (ksi)	h/4 (in)	Abrst_rqrd (in2)
2548.66	20.00	16.50	5.10



BR01 - Vertical Shear, Span 1, Beam 2, STRENGTH I

HORIZONTAL SHEAR (Art. 5.8.4) - Span : 1, Beam : 2
(Beam and Slab effects are INCLUDED in Vu).
Computed Interface width considered to be engaged in shear transfer, bvi = 48.00(in).

Location (ft) / Vu (kips)	Vnh-req (kips/in)	de (in)	a (in)	dv (in)	s_max (in)	Avh-min (in2/ft)	Avh-sm (in2/ft)	Avh-rg (in2/ft)	Avh-prvd (in2/ft)
Bearing : 458.6	0.00 7.61	67.69	1.48	66.94	24.00	0.480	1.337	0.000	1.600
Transfer : 446.3	2.17 7.62	67.69	5.27	65.05	24.00	0.480	1.341	0.000	0.800*
Critical : 427.9	5.38 7.36	67.69	6.16	64.61	24.00	0.480	1.253	0.000	0.229*
0.1L :	13.14								

SAY OK, SURFACE IS ROUGHENED

Location (ft) / Vu (kips)	Vnh-req (kips/in)	de (in)	a (in)	dv (in)	s_max (in)	Avh-min (in2/ft)	Avh-sm (in2/ft)	Avh-rg (in2/ft)	Avh-prvd (in2/ft)
383.8	6.70	67.69	8.17	63.60	24.00	0.480	1.035	0.000	0.229*
0.2L : 307.0	27.11 5.82	62.75	8.21	58.65	24.00	0.480	0.739	0.000	0.229*
0.3L : 230.6	41.09 4.37	62.72	8.22	58.61	24.00	0.480	0.257	0.000	0.229*
0.4L : 156.4	55.06 2.82	65.66	8.23	61.54	24.00	0.480	0.000	0.000	0.229
0.5L : 83.8	69.04 1.48	67.12	8.23	63.01	24.00	0.480	0.000	0.000	0.229
0.6L : 156.4	83.01 2.82	65.66	8.23	61.54	24.00	0.480	0.000	0.000	0.229
0.7L : 230.6	96.98 4.37	62.72	8.22	58.61	24.00	0.480	0.257	0.000	0.229*
0.8L : 307.0	110.96 5.82	62.75	8.21	58.65	24.00	0.480	0.739	0.000	0.229*
0.9L : 383.8	124.93 6.70	67.69	8.17	63.60	24.00	0.480	1.035	0.000	0.229*
Critical : 427.9	132.69 7.36	67.69	6.16	64.61	24.00	0.480	1.253	0.000	0.229*
Transfer : 446.3	135.90 7.62	67.69	5.27	65.05	24.00	0.480	1.341	0.000	0.800*
Bearing : 458.6	138.07 7.61	67.69	1.48	66.94	24.00	0.480	1.337	0.000	1.600

SAY OK, SURFACE IS ROUGHENED

CAMBER/DEFLECTION

CAMBER AND DEFLECTIONS: SERVICE I
(Span : 1, Beam : 2; Units: in)

	Release	Mult	Erection	Mult	Final
At 0.1 x L =	13.14 ft				
Prestress	2.186	1.80	3.935	2.20	4.810
Self Wt.	-0.946	1.85	-1.749	2.40	-2.270
Deck + Haunch			-0.590	2.30	-1.357
DL-Prec. (DC)			-0.186	3.00	-0.557
Diaphragm			-0.011	3.00	-0.032
DL-Prec. (DW)			0.000	3.00	0.000
DL-Comp. (DC)			-0.054	3.00	-0.163
DL-Comp. (DW)			-0.201	3.00	-0.602
Live Load					-0.291
Total	1.241		1.145		-0.461

	Release	Mult	Erection	Mult	Final
At 0.2 x L =	27.11 ft				
Prestress	4.002	1.80	7.204	2.20	8.805
Self Wt.	-1.789	1.85	-3.310	2.40	-4.294
Deck + Haunch			-1.153	2.30	-2.651
DL-Prec. (DC)			-0.362	3.00	-1.087
Diaphragm			-0.021	3.00	-0.063
DL-Prec. (DW)			0.000	3.00	0.000
DL-Comp. (DC)			-0.106	3.00	-0.318
DL-Comp. (DW)			-0.392	3.00	-1.177
Live Load					-0.573
Total	2.213		1.860		-1.358

	Release	Mult	Erection	Mult	Final
At 0.3 x L =	41.09 ft				
Prestress	5.380	1.80	9.683	2.20	11.835
Self Wt.	-2.449	1.85	-4.531	2.40	-5.879
Deck + Haunch			-1.594	2.30	-3.667
DL-Prec. (DC)			-0.501	3.00	-1.504
Diaphragm			-0.029	3.00	-0.087
DL-Prec. (DW)			0.000	3.00	0.000
DL-Comp. (DC)			-0.147	3.00	-0.440
DL-Comp. (DW)			-0.543	3.00	-1.628
Live Load					-0.801
Total	2.930		2.338		-2.170

	Release	Mult	Erection	Mult	Final
At 0.4 x L =	55.06 ft				

	Release	Mult	Erection	Mult	Final
Prestress	6.251	1.80	11.252	2.20	13.753
Self Wt.	-2.869	1.85	-5.307	2.40	-6.885
Deck + Haunch			-1.875	2.30	-4.313
DL-Prec. (DC)			-0.590	3.00	-1.769
Diaphragm			-0.034	3.00	-0.103
DL-Prec. (DW)			0.000	3.00	0.000
DL-Comp. (DC)			-0.172	3.00	-0.517
DL-Comp. (DW)			-0.638	3.00	-1.915
Live Load					-0.950
Total	3.382		2.635		-2.698

	Release	Mult	Erection	Mult	Final
At 0.5 x L =	69.04 ft				
Prestress	6.552	1.80	11.794	2.20	14.415
Self Wt.	-3.012	1.85	-5.573	2.40	-7.230
Deck + Haunch			-1.971	2.30	-4.534
DL-Prec. (DC)			-0.620	3.00	-1.860
Diaphragm			-0.036	3.00	-0.108
DL-Prec. (DW)			0.000	3.00	0.000
DL-Comp. (DC)			-0.181	3.00	-0.544
DL-Comp. (DW)			-0.671	3.00	-2.013
Live Load					-0.995
Total	3.540		2.742		-2.868

	Release	Mult	Erection	Mult	Final
At 0.6 x L =	83.01 ft				
Prestress	6.251	1.80	11.252	2.20	13.753
Self Wt.	-2.869	1.85	-5.307	2.40	-6.885
Deck + Haunch			-1.875	2.30	-4.313
DL-Prec. (DC)			-0.590	3.00	-1.769
Diaphragm			-0.034	3.00	-0.103
DL-Prec. (DW)			0.000	3.00	0.000
DL-Comp. (DC)			-0.172	3.00	-0.517
DL-Comp. (DW)			-0.638	3.00	-1.915
Live Load					-0.950
Total	3.382		2.635		-2.698

	Release	Mult	Erection	Mult	Final
At 0.7 x L =	96.98 ft				
Prestress	5.380	1.80	9.683	2.20	11.835
Self Wt.	-2.449	1.85	-4.531	2.40	-5.879
Deck + Haunch			-1.594	2.30	-3.667
DL-Prec. (DC)			-0.501	3.00	-1.504
Diaphragm			-0.029	3.00	-0.087
DL-Prec. (DW)			0.000	3.00	0.000
DL-Comp. (DC)			-0.147	3.00	-0.440
DL-Comp. (DW)			-0.543	3.00	-1.628

	Release	Mult	Erection	Mult	Final
Live Load					-0.801
Total	2.930		2.338		-2.170

	Release	Mult	Erection	Mult	Final
At 0.8 x L =	110.96 ft				
Prestress	4.002	1.80	7.204	2.20	8.805
Self Wt.	-1.789	1.85	-3.310	2.40	-4.294
Deck + Haunch			-1.153	2.30	-2.651
DL-Prec. (DC)			-0.362	3.00	-1.087
Diaphragm			-0.021	3.00	-0.063
DL-Prec. (DW)			0.000	3.00	0.000
DL-Comp. (DC)			-0.106	3.00	-0.318
DL-Comp. (DW)			-0.392	3.00	-1.177
Live Load					-0.573
Total	2.213		1.860		-1.358

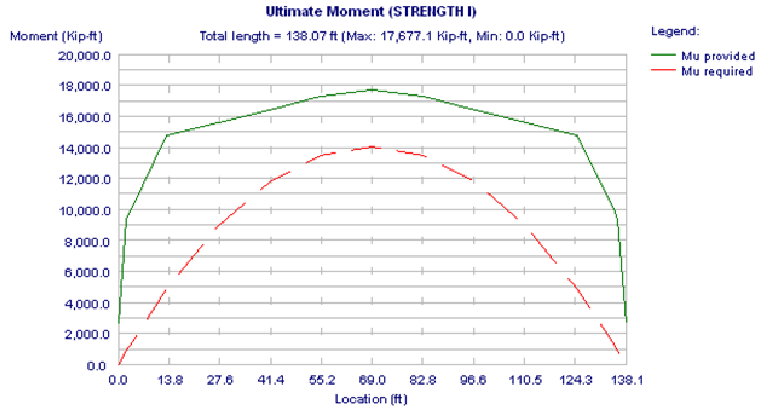
	Release	Mult	Erection	Mult	Final
At 0.9 x L =	124.93 ft				
Prestress	2.186	1.80	3.935	2.20	4.810
Self Wt.	-0.946	1.85	-1.749	2.40	-2.270
Deck + Haunch			-0.590	2.30	-1.357
DL-Prec. (DC)			-0.186	3.00	-0.557
Diaphragm			-0.011	3.00	-0.032
DL-Prec. (DW)			0.000	3.00	0.000
DL-Comp. (DC)			-0.054	3.00	-0.163
DL-Comp. (DW)			-0.201	3.00	-0.602
Live Load					-0.291
Total	1.241		1.145		-0.461

ULTIMATE MOMENT

ULTIMATE - Span : 1, Beam : 2, STRENGTH I
(Mr-prvd computed by Strain Compatibility method. Ult. Conc. Strain = 0.00300)

Location (ft) Mu k.ft	dp in	Aps in2	fps ksi	c in	a in	Mr-prvd k.ft	c/dt	Phi	1.2 Mcr k.ft	min Mr k.ft	Crkg Ratio	Mu-p/r Ratio
Transfer	2.17											
877.0	54.9	8.027	266.1	6.4	5.3	9307.3	0.089T	1.00	-	-	-	-
H/2	3.06											
1232.2	55.1	8.467	266.0	6.7	5.6	9821.2	0.094T	1.00	-	-	-	-
0.1L	13.14											
4886.2	57.1	12.586	265.6	9.9	8.2	14761.9	0.138T	1.00	11501.9	6498.6	1.5	-
0.2L	27.11											
8936.9	59.9	12.586	266.9	9.9	8.2	15597.8	0.139T	1.00	11781.1	11781.1	1.6	-
0.3L	41.09											
11799.2	62.7	12.586	267.3	9.9	8.2	16430.2	0.139T	1.00	12218.3	12218.3	1.6	-
0.4L	55.06											
13506.2	65.7	12.586	267.6	9.9	8.2	17261.5	0.139T	1.00	12813.5	12813.5	1.6	-
0.5L	69.04											
14047.9	67.1	12.586	267.6	9.9	8.2	17677.1	0.139T	1.00	13149.5	13149.5	1.6	-
0.6L	83.01											
13506.2	65.7	12.586	267.6	9.9	8.2	17261.5	0.139T	1.00	12813.5	12813.5	1.6	-
0.7L	96.98											
11799.2	62.7	12.586	267.3	9.9	8.2	16430.2	0.139T	1.00	12218.3	12218.3	1.6	-
0.8L	110.96											
8936.9	59.9	12.586	266.9	9.9	8.2	15597.8	0.139T	1.00	11781.1	11781.1	1.6	-
0.9L	124.93											
4886.2	57.1	12.586	265.6	9.9	8.2	14761.9	0.138T	1.00	11501.9	6498.6	1.5	-
H/2	135.01											
1232.2	55.1	8.467	266.0	6.7	5.6	9821.2	0.094T	1.00	-	-	-	-
Transfer	135.90											
877.0	54.9	8.027	266.1	6.4	5.3	9307.3	0.089T	1.00	-	-	-	-

Legend: C = Compression-Controlled (c/dt > 0.600)
I = In-Transition (0.60 >= c/dt > 0.375)
T = Tension-Controlled (c/dt <= 0.375)
Note : fr used for calculating Mcr is computed using AASHTO method (Art.5.4.2.6.)
Consider Bottom Tension Steel Contribution : NO



BR01 - Ultimate Moment, Span 1, Beam 2, STRENGTH I

DETENSIONING

Span : 1, Beam : 2; Groups 1-32; Units: ksi

Grp	Str		Ys,in		3.00ft
1	2	E	2.00	Ft	0.082
		M	2.00	Fb	0.082
2	2	E	63.00	Ft	0.295
		M	12.00	Fb	0.032
3	1	E	63.00	Ft	0.401
		M	12.00	Fb	0.007
4	2	E	61.00	Ft	0.604
		M	10.00	Fb	-0.033
5	1	E	61.00	Ft	0.706
		M	10.00	Fb	-0.054
6	2	E	59.00	Ft	0.899
		M	8.00	Fb	-0.084
7	1	E	59.00	Ft	0.996
		M	8.00	Fb	-0.099
8	2	E	57.00	Ft	1.180
		M	6.00	Fb	-0.120
9	1	E	57.00	Ft	1.272
		M	6.00	Fb	-0.131
10	2	E	55.00	Ft	1.446
		M	4.00	Fb	-0.142
11	1	E	55.00	Ft	1.533
		M	4.00	Fb	-0.148
12	2	E	12.00	Ft	1.514
		M	12.00	Fb	0.036
13	2	E	10.00	Ft	1.486
		M	10.00	Fb	0.230
14	2	E	10.00	Ft	1.457
		M	10.00	Fb	0.424
15	2	E	10.00	Ft	1.429
		M	10.00	Fb	0.618
16	2	E	8.00	Ft	1.391
		M	8.00	Fb	0.822
17	2	E	8.00	Ft	1.353
		M	8.00	Fb	1.026
18	2	E	8.00	Ft	1.315
		M	8.00	Fb	1.229
19	2	E	8.00	Ft	1.277
		M	8.00	Fb	1.433
20	2	E	6.00	Ft	1.229
		M	6.00	Fb	1.646
21	2	E	6.00	Ft	1.182
		M	6.00	Fb	1.859
22	2	E	6.00	Ft	1.134
		M	6.00	Fb	2.073

Grp	Str		Ys,in		3.00ft
23	2	E	6.00	Ft	1.087
		M	6.00	Fb	2.286
24	2	E	4.00	Ft	1.030
		M	4.00	Fb	2.509
25	2	E	4.00	Ft	0.973
		M	4.00	Fb	2.732
26	2	E	4.00	Ft	0.916
		M	4.00	Fb	2.955
27	2	E	4.00	Ft	0.859
		M	4.00	Fb	3.178
28	2	E	2.00	Ft	0.792
		M	2.00	Fb	3.410
29	2	E	2.00	Ft	0.725
		M	2.00	Fb	3.643
30	2	E	2.00	Ft	0.659
		M	2.00	Fb	3.875
31	2	E	2.00	Ft	0.592
		M	2.00	Fb	4.108
32	1	E	2.00	Ft	0.559
		M	2.00	Fb	4.224

DESIGN SUMMARY

Span: 1, Beam: 2, Interior beam

Beam type:	I-Girder,	ODOT 66" - 4ft - SHOP
Precast Length,	ft	139.74
Release Length,	ft	139.74
Strand Pattern:	Straight/Draped	Depr. Point: 0.45 L
Strand:	6/10-270K-LL	
Strand Es,	ksi:	28500.0
No. of strands:	58	
	Draped:	15
	Straight:	43
Concrete Strength:		
	f'ci:	7.5 ksi
	f'c:	11.0 ksi
	f'ct:	4.5 ksi
Initial losses:	10.71 %	
Final losses:	15.70 %	

Specification	Allowable	Computed	Location	Status
Release Stresses (ksi) (Art. 5.9.4.1)				
Precast Bot (compression)	4.500	4.357	Depress	OK
Precast Top w/ no reinf. (tension)	-0.200	0.446	Midspan	
Precast Top w/ reinf. (tension)	-0.657			
Strength I (Art. 3.4.1, 5.7.3.1.1)	Provided	Required	Location	Status
Ult. Moment (k.ft)	17677.06	14047.94	Midspan	OK
Debonding Limits (Art. 5.11.4.3)	Allowable	Computed		Status
Max. Debond per Row	40.00 %	0.00 %		OK
Max. Debond Total	25.00 %	0.00 %		OK

Positive Moment Envelope Stresses (ksi) (Art. 3.4.1 and 5.9.4.2)

Specification	Allow	Final 1 Comp	Loc.	Allow	Final 2 Comp	Loc.	Allow	Final 3 Comp	Loc.
Service I Limit State - Compressive	Stresses	Only							
Precast Top	6.600	3.551	Midspan	4.950	2.801	0.4L/0.6L			
Precast Bot	6.600	3.737	Transfer	4.950	3.849	Transfer			
Service III Limit State - Tensile	Stresses	Only							
Precast Top	-0.314	0.124	Bearing						
Precast Bot	-0.314	-0.186	Midspan						
Fatigue I Limit State - Compressive	Stresses	Only							
Precast Top							4.400	1.725	0.4L/0.6L
Precast Bot							4.400	1.874	Transfer

CAMBER / DEFLECTION: (PCI Design Handbook - 4th Ed.- Table 4.6.2)
0.5 x L = 69.04 ft

	Release	Mult	Erection	Mult	Final
Prestress	6.552	1.80	11.794	2.20	14.415
Self Wt.	-3.012	1.85	-5.573	2.40	-7.230
Deck + Haunch			-1.971	2.30	-4.534
DL-Prec. (DC)			-0.620	3.00	-1.860
Diaphragm			-0.036	3.00	-0.108
DL-Prec. (DW)			0.000	3.00	0.000
DL-Comp. (DC)			-0.181	3.00	-0.544
DL-Comp. (DW)			-0.671	3.00	-2.013
Live Load					-0.995
Total	3.540		2.742		-2.868

Positive values indicate upward deflection.

ABUTMENT DESIGN

Calculations For	Innerbelt Bridge No. 6	Job No.	49633	Sheet No.
Made by	AKS	Date	4/20/2011	
Checked by	DBT	Date	4/22/2011	
Backchecked by	AKS	Date	4/25/2011	

HNTB

TITLE: Abutment Design Approach

Design Code:

- AASHTO LRFD Bridge Design Specifications for Highway Bridges, 4th Edition - 2007, Interims through 2009
- ODOT LRFD BDM, dated 04-16-2010

Structure Type Description:

- Semi-integral abutment on HP 16x88 Steel H Piles
- Elastomeric bearings
- At the Forward Abutment there is an existing 66" Sanitary Sewer Pipe that is not being relocated. At this location the abutment design was split up into two different segments:

The first was a "typical" length, going from the middle of Beams 3 and 4 to the right end of the Forward Abutment.

The second "over the pipe" length is from the left end of the Forward Abutment to the middle of Beams 3 and 4. The spacing between piles on either side of the pipe exceed the maximum spacing per ODOT BDM 303.4.2.2, to make up for this, additional longitudinal steel was designed for this section of footing spanning over the pipe.

Pile Layout:

- As a minimum per ODOT BDM 303.2.2.7 two rows of piles were considered. Battered piles will not be an option at this location due to the MSE walls.

Pile Capacity:

- A Draft Geotech Memo was prepared, see sheets at end of design calculations for unfactored Estimated Axial Pile Resistances in the soil. A structural capacity check is also included.

Analysis:

Calculations: All calculations for the piles and additional reinforcing are performed within this spreadsheet.

Piles: HP 16x88 Steel H Piles

Live Load Surcharge:
Simulate live load surcharge with an equivalent height of soil in accordance with AASHTO 3.11.6.4.

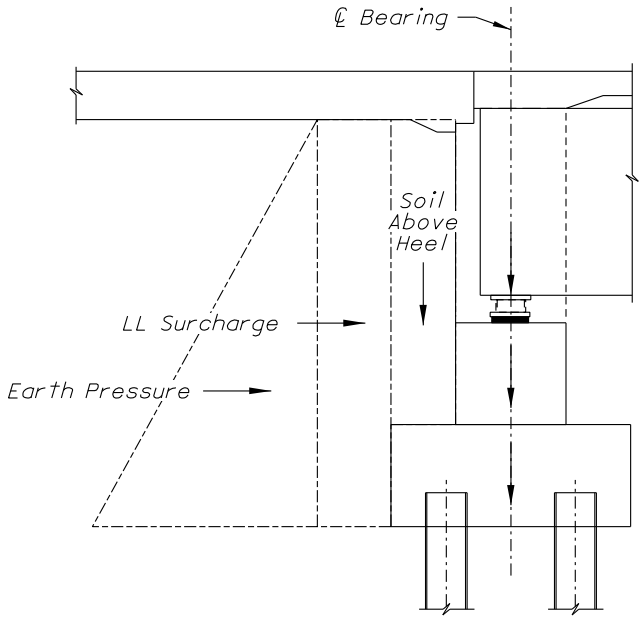
Lateral Earth Pressure:
Equivalent Fluid Pressure (EPF) was calculated based on the project document GD-2, "Recommended Default Soil and Rock Parameters for Design and Analysis" and AASHTO 3.11.5.3 equations. A Final Geotech report is not yet available for this design build project. This value must be checked against the final Geotech report when available.

Load Cases: Reference AASHTO Table 3.4.1-1.
Strength I: Final condition
Strength I: Construction
Extreme Event I

Note: Horizontal Superstructure forces will not be considered for this abutment.

Calculations For	Innerbelt Bridge No. 6	Job No.	49633	Sheet No.
Made by	AKS	Date	4/20/2011	
Checked by	DBT	Date	4/22/2011	
Backchecked by	AKS	Date	4/25/2011	

TITLE: Equivalent Fluid Pressure



Angle of Back Face of Wall to the horizontal, θ =	90 deg.	*1.571 rad*	
Effective angle of Internal Friction, φ'_f =	34 deg.	*0.593 rad*	(Geotech Report)
Friction angle between fill and wall, δ =	29 deg.	*0.506 rad*	(AASHTO Table 3.11.5.3-1)
Angle of Fill to the horizontal, β =	0 deg.	*0.000 rad*	

$$\Gamma = [1 + \sqrt{\frac{\sin(\phi'_f + \delta)\sin(\phi'_f - \beta)}{\sin(\theta - \delta)\sin(\theta + \beta)}}]^2 =$$ 3.079 AASHTO Eq. 3.11.5.3-2

$$k_a = \frac{\sin^2(\theta + \phi'_f)}{\Gamma[\sin^2\theta \sin(\theta - \delta)]} =$$ 0.255 AASHTO Eq. 3.11.5.3-1

Soil Unit Weight, γ_t =	125 pcf	(Geotech Report, Table GD-2.1, Common Borrow)
Equivalent Fluid Pressure, EFP = ka*γ_t =	31.9 pcf	

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TITLE: Pile Capacity

- Assumes no Pile Tips

Pile Type = H-Piles

φ_c = 0.6

φ_{dyn} = 0.65

Pile Size	Fy (ksi)	Area, A ($in.^2$)	Structural Capacity $R_n = \varphi_c FyA$ (kip)	Geotech Capacity $R_r = \varphi_{dyn} Rn$ (kip)
HP 16x88	50	25.8	774.0	503.1

Use 235 kips @ Rear Abutment *
234 kips @ Forward Abutment (Typical Length) *
250 kips @ Forward Abutment (Length over Pipe) *

* Due to pile capacity charts and 100' depth limit. (Due to consolidation concerns from unconsolidated layer at 100') See Geotechnical Memo.

Calculations For	Innerbelt Bridge No. 6	Job No.	49633	Sheet No.
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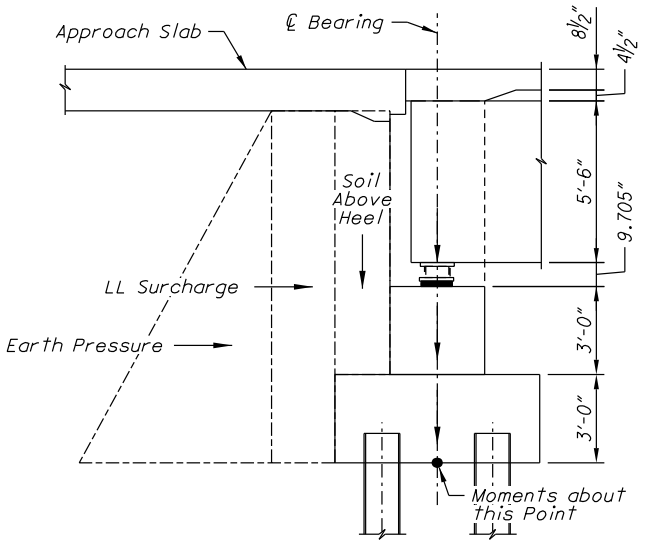
TITLE: Rear Abutment Pile Design

NOTE: Input data is denoted by shading. All other values are calculations performed by the spreadsheet.

REFERENCE: 2007 AASHTO LRFD, 4th Edition, with Interims Through 2009



Vertical Loads & Moments

Item	Height (ft)	Width (ft)	Length (ft)	Weight (kips)	Moment Arm (ft)	Moment About CL Ftg (k-ft)	Note: Moments are taken about the CL of footing and if arm is to the left of CL footing, per the figures above, input as a negative value.
Backwall	0.000	0.000	0.000	0.00	0.000	0.00	
Cap	3.000	3.000	86.302	116.51	0.000	0.00	
Footing	3.000	6.500	86.302	252.43	0.000	0.00	
Soil on Heel	8.975	1.750	86.302	162.67	2.375	386.33	
Approach Slab	1.417	30.000	86.302	275.09	1.250	343.86	←Assume 1/2 is distributed to abut.
Wings	-	0.000	-	0.00	0.000	0.00	MSE Wall in lieu of Wingwall
Substructure DL				806.69		730.19	

	Number of Beams	10					
	Ext. Beam Rxn (kips)	Int. Beam Rxn (kips)					
DL Superstructure, DC	154.3	155.2		1550.20	0.000	0.00	Rxn = Self Wt + DL Prec DC + Deck+Haunch + Diaph
DL Super. FWS, DW	32.50	35.50		349.00	0.000	0.00	Rxn = DL Comp DW
Superstructure LL (no Impact)	96.09	105.57		1036.70	0.000	0.00	Rxn = LL+I / 1.15
Pedestrian LL	0.00	0.00		0.00	0.000	0.00	
	Height (ft)	Width (ft)	Length (ft)				
End Diaphragm	6.684	3.000	86.302	259.57	0.000	0.00	Height = haunch + beam + bearing

Horizontal Loads & Moments

Earth Pressure:

Abutment Type =	Semi-Integral	
Equivalent Fluid Pressure, EFP =	31.90 pcf	
Length =	86.30 ft	
Pressure @ Top of Non-Int. Backwall or @ Top of Semi-Int. Cap, P_t =	213.22 psf	
Pressure @ Bottom of Cap or Footing, P_b =	404.62 psf	
$P_{E1} = P_t$ x (Cap + Ftg Height) x Length =	110.41 kips	Values greater than "0" if a semi-integral abutment
M_{E1} = 1/2 x (Cap + Ftg Height) x P_{E1} =	331.22 kip-ft	is used
P_{E2} = 1/2 x (P_b-P_t) x (Backwall + Cap +Ftg Ht.) x Length =	37.86 kips	
M_{E2} = 1/3 x (Backwall + Cap +Ftg Ht.) x P_{E2} =	57.83 kip-ft	
P_{ETotal} =	148.26 kips	
M_{ETotal} =	389.05 kip-ft	

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TITLE: Rear Abutment Pile Design

NOTE: Input data is denoted by shading. All other values are calculations performed by the spreadsheet.

REFERENCE: 2007 AASHTO LRFD, 4th Edition, with Interims Through 2009

LL Surcharge:

Is there an approach slab, Yes or No?	No	If "Yes", then values in final condition design are zeroed out.
P_S = 2ft x EFP x Height x Length =	33.04 kips	**Say "No" since ODOT BDM S3.11.6.5 states that no reduction should be applied even with the presence of an approach slab.**
M_E = 1/2 x Height x P_S =	99.11 kip-ft	

Temperature:

Temperature Rise/Fall, T =	60 degrees	Determined using a 60° median temperature and Table 3.12.2.1-1
Coefficient of Expansion and Contraction, C_{EC} =	0.000006	per degree Fahrenheit (AASHTO 5.4.2.2 for Concrete & 6.4.1 for Steel)
Length Contributing to Temperature Movement, L_T =	0.000 ft	**Do not consider temperature movement for semi-integral abutment per ODOT BDM 306.3.1**
Δ_T = T x C_{EC} x L_T x 12 in/ft =	0.000 in	
Bearing Pad Width, P_W =	20.000 in	
Bearing Pad Length, P_L =	12.000 in	
Bearing Pad Thickness, P_T =	1.844 in	Elastomer Thickness Only
Bearing Pad Shear Modulus, G =	130 psi	Use highest value within range for Durometer Hardness 50 to maximize load
Number of Pads per Beam, n_P =	1	
Distance Top of Cap to Bottom of Footing, D_{CF} =	6.000 ft	
P_{Temp} = Δ_T x P_W x P_L x G x n_P x No. Beams / P_T =	0.00 kips	
M_T = P_{Temp} x D_{CF} =	0.00 kip-ft	

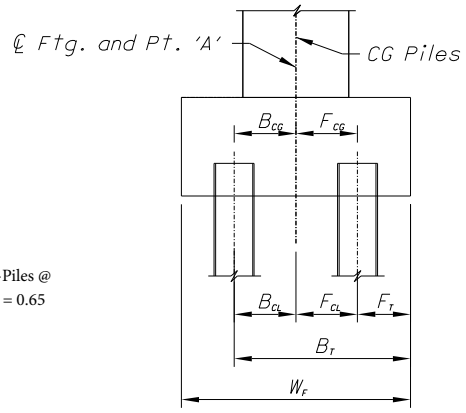
Earthquake:

Note: Per AASHTO 3.10.9.2 take 25% of the vertical reaction (100% to be applied in longitudinal direction only).

	Vertical Reaction	Lateral Load	Arm	Moment
DL Superstructure, DC	1550.20 kips	387.55 kips	3.000 ft	1162.65 kip-ft
End Diaphragm	259.57 kips	64.89 kips	3.000 ft	194.68 kip-ft
Superstructure LL (50% During Event)	518.35 kips	129.59 kips	3.000 ft	388.76 kip-ft
		582.03 kips		1746.09 kip-ft

Footing and Pile Input

Footing Width, W_F =	6.500 ft	
Distance from Front Pile to Toe, F_T =	1.50 ft	
Distance from Back Pile to Toe, B_T =	5.000 ft	
Distance from Front Pile to CL Footing, F_{CL} =	1.750 ft	
Distance from Back Pile to CL Footing, B_{CL} =	1.750 ft	
Number of Front Piles, n_F =	14	
Number of Back Piles, n_B =	14	
Pile Size or Diameter =	15.7 in	
Maximum Pile Spacing =	8.000 ft	per ODOT BDM 303.4.2.2
Anticipated Front Row Pile Spacing =	6.408 ft	**OK!**
Anticipated Back Row Pile Spacing =	6.408 ft	**OK!**
Factored Nominal Pile Axial Resistance =	235.30 kips	← 362 kip, HP16x88 Steel H-Piles @ 91' depth per pile curves, Φ = 0.65
Distance from 'A' to CG Piles, A_{CG} =	0.000 ft	
Distance from Front Pile to CG, F_{CG} =	1.750 ft	
Distance from Back Pile to CG, B_{CG} =	1.750 ft	
nd^2 = n_F x F_{CG}^2 + n_B x B_{CG}^2 =	85.750	
Are the front row of piles battered, Yes or No?	No	
Downdrag Force =	0.00 kips	None anticipated per Geotech



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TITLE: Rear Abutment Pile Design

NOTE: Input data is denoted by shading. All other values are calculations performed by the spreadsheet.

REFERENCE: 2007 AASHTO LRFD, 4th Edition, with Interims Through 2009

Strength I - Final Condition Case

Load Factors (AASHTO Table 3.4.1-1 & 3.4.1-2)

For DC Dead Loads, γ_{pDC} =	1.25
For FWS Load (DW Max), γ_{pDW} =	1.50
For Live Loads, γ_{LL} =	1.75
For Downdrag, γ_{pDD} =	1.40
For Earth Pressure, γ_{pEH} =	1.50
For Earth LL Surcharge, γ_{pES} =	1.50
For Uniform Temperature, γ_{TU} =	1.20

Summary of Factored Loads

	Vertical Forces			Horizontal Forces	
	P_V	M_A		P_H	M_A
DL Abutment	1008.37 kips	912.74 kip-ft	Earth Pressure	222.39 kips	583.58 kip-ft
DL Superstructure, DC	2262.21 kips	0.00 kip-ft	LL Surcharge	49.56 kips	148.67 kip-ft
DL Super. FWS, DW Max	523.50 kips	0.00 kip-ft	Temperature	0.00 kips	0.00 kip-ft
Superstructure LL	1814.22 kips	0.00 kip-ft			
Pedestrian LL	0.00 kips	0.00 kip-ft			
Downdrag	0.00 kips	0.00 kip-ft			

Total Vertical Forces, P_V =	5608.30 kips	Does not include downdrag		
Total Horizontal Forces, P_H =	271.95 kips			
Total Moments, M_A =	1644.99 kip-ft			
$M_{CG} = M_A - P_V \times A_{CG}$ =	1644.99 kip-ft			
$F_{VFront} = P_V / (n_F + n_B) + M_{CG} \times F_{CG} / nd^2$ =	233.87 kips			
$F_{HFront} = F_{VFront} \times {}^4/_{12}$ Batter (or = 0 without batter) =	0.00 kips	= 0 kips if no batter on front row of piles		
Net Horizontal = $(P_H - n_F \times F_{HFront}) / n_{Total}$ =	9.71 kips	= P_h / Total # piles if no batter on front row of piles		
$Axial_{Front} = ({}^{12.6491}/_{12}) \times F_{VFront}$ (with batter or = F_{VFront} w/out) =	233.87 kips	**OK**	with downdrag =	233.87 kips **OK**
$Axial_{Back} = P_V / n_{Total} - M_{CG} \times B_{CG} / nd^2$ =	166.73 kips	**OK**	with downdrag =	166.73 kips **OK**

Strength I - Construction Condition Case

Includes LL surcharge but does not include superstructure LL or future wearing surface.

Load Factors (AASHTO Table 3.4.1-1 & 3.4.1-2)

For DC Dead Loads, γ_{pDC} =	1.25
For FWS Load (DW Max), γ_{pDW} =	1.50
For Live Loads, γ_{LL} =	1.75
For Downdrag, γ_{pDD} =	1.40
For Earth Pressure, γ_{pEH} =	1.50
For Earth LL Surcharge, γ_{pES} =	1.50
For Uniform Temperature, γ_{TU} =	1.20

Summary of Factored Loads

	Vertical Forces			Horizontal Forces	
	P_V	M_A		P_H	M_A
DL Abutment	1008.37 kips	912.74 kip-ft	Earth Pressure	222.39 kips	583.58 kip-ft
DL Superstructure, DC	2262.21 kips	0.00 kip-ft	LL Surcharge	57.82 kips	173.45 kip-ft
DL Super. FWS, DW Max	0.00 kips	0.00 kip-ft	Temperature	0.00 kips	0.00 kip-ft
Superstructure LL	0.00 kips	0.00 kip-ft			
Pedestrian LL	0.00 kips	0.00 kip-ft			
Downdrag	0.00 kips	0.00 kip-ft			

Total Vertical Forces, P_V =	3270.58 kips	Does not include downdrag		
Total Horizontal Forces, P_H =	280.21 kips			
Total Moments, M_A =	1669.76 kip-ft			
$M_{CG} = M_A - P_V \times A_{CG}$ =	1669.76 kip-ft			
$F_{VFront} = P_V / (n_F + n_B) + M_{CG} \times F_{CG} / nd^2$ =	150.88 kips			
$F_{HFront} = F_{VFront} \times {}^4/_{12}$ Batter (or = 0 without batter) =	0.00 kips	= 0 kips if no batter on front row of piles		
Net Horizontal = $(P_H - n_F \times F_{HFront}) / n_{Total}$ =	10.01 kips	= P_h / Total # piles if no batter on front row of piles		
$Axial_{Front} = ({}^{12.6491}/_{12}) \times F_{VFront}$ (with batter or = F_{VFront} w/out) =	150.88 kips	**OK**	with downdrag =	150.88 kips **OK**
$Axial_{Back} = P_V / n_{Total} - M_{CG} \times B_{CG} / nd^2$ =	82.73 kips	**OK**	with downdrag =	82.73 kips **OK**

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Backchecked by	AKS	Date	4/25/2011	1/16/2012

TITLE: Rear Abutment Pile Design

NOTE: Input data is denoted by shading. All other values are calculations performed by the spreadsheet.

REFERENCE: 2007 AASHTO LRFD, 4th Edition, with Interims Through 2009

Exreme Event I

Load Factors (AASHTO Table 3.4.1-1 & 3.4.1-2)

For DC Dead Loads, γ_{pDC} =	1.25
For FWS Load (DW Max), γ_{pDW} =	0.00
For Live Loads, γ_{LL} =	1.00
For Downdrag, γ_{pDD} =	1.40
For Earth Pressure, γ_{EH} =	0.00
For Earthquake, γ_{EQ} =	1.00

Summary of Factored Loads

	Vertical Forces			Horizontal Forces	
	P_V	M_A		P_H	M_A
DL Abutment	1008.37 kips	912.74 kip-ft	Earth Pressure	0.00 kips	0.00 kip-ft
DL Superstructure, DC	2262.21 kips	0.00 kip-ft	LL Surcharge	33.04 kips	99.11 kip-ft
DL Super FWS, DW Max	0.00 kips	0.00 kip-ft	Earthquake	582.03 kips	1746.09 kip-ft
Superstructure Live Load	0.00 kips	0.00 kip-ft			
Downdrag	0.00 kips	0.00 kip-ft			

Total Vertical Forces, P_V =	3270.58 kips				
Total Horizontal Forces, P_H =	615.07 kips				
Total Moments, M_A =	2757.94 kip-ft				
$M_{CG} = M_A - P_V \times A_{CG}$ =	2757.94 kip-ft				
$F_{VFront} = P_V / (n_F + n_B) + M_{CG} \times F_{CG} / nd^2$ =	173.09 kips				
$F_{HFront} = F_{VFront} \times {}^{4}/_{12}$ Batter (or = 0 without batter) =	0.00 kips	= 0 kips if no batter on front row of piles			
Net Horizontal = $(P_H - n_F \times F_{HFront}) / n_{Total}$ =	21.97 kips	= P_h / Total # piles if no batter on front row of piles			
$Axial_{Front} = ({}^{12.6491}/_{12}) \times F_{VFront}$ (with batter or = F_{VFront} w/out) =	173.09 kips	**OK**	with downdrag =	173.09 kips	**OK**
$Axial_{Back} = P_V / n_{Total} - M_{CG} \times B_{CG} / nd^2$ =	60.52 kips	**OK**	with downdrag =	60.52 kips	**OK**

Calculations For	Innerbelt Bridge No. 6	Job No.	49633	Sheet No.
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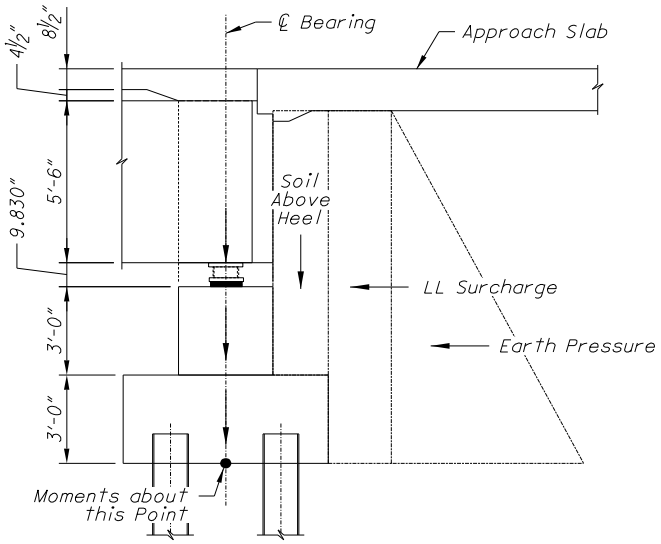
TITLE: Forward Abutment Pile Design

NOTE: Input data is denoted by shading. All other values are calculations performed by the spreadsheet.

REFERENCE: 2007 AASHTO LRFD, 4th Edition, with Interims Through 2009



Vertical Loads & Moments

Item	Height (ft)	Width (ft)	Length (ft)	Weight (kips)	Moment Arm (ft)	Moment About CL Ftg (k-ft)	Note: Moments are taken about the CL of footing and if arm is to the left of CL footing, per the figures above, input as a negative value.
Backwall	0.000	0.000	0.000	0.00	0.000	0.00	
Cap	3.000	3.000	62.333	84.15	0.000	0.00	
Footing	3.000	6.500	62.333	182.33	0.000	0.00	
Soil on Heel	8.986	1.750	62.333	117.62	2.375	279.36	
Approach Slab	1.417	30.000	62.333	198.69	1.250	248.36	←Assume 1/2 is distributed to abut.
Wings	-	0.000	-	0.00	0.000	0.00	MSE Wall in lieu of Wingwall
Substructure DL				582.79		527.72	

	Number of Beams	7					
	Ext. Beam Rxn (kips)	Int. Beam Rxn (kips)					
DL Superstructure, DC	154.3	155.2		1084.60	0.000	0.00	Rxn = Self Wt + DL Prec DC + Deck+Haunch + Diaph
DL Super. FWS, DW	32.50	35.50		242.50	0.000	0.00	Rxn = DL Comp DW
Superstructure LL (no Impact)	96.09	105.57		720.00	0.000	0.00	Rxn = LL+I / 1.15
Pedestrian LL	0.00	0.00		0.00	0.000	0.00	
	Height (ft)	Width (ft)	Length (ft)				
End Diaphragm	6.694	3.000	62.333	187.77	0.000	0.00	Height = haunch + beam + bearing

Horizontal Loads & Moments

Earth Pressure:

Abutment Type =	Semi-Integral	
Equivalent Fluid Pressure, EFP =	31.90 pcf	
Length =	62.33 ft	
Pressure @ Top of Non-Int. Backwall or @ Top of Semi-Int. Cap, P_t =	213.55 psf	
Pressure @ Bottom of Cap or Footing, P_b =	404.95 psf	
$P_{E1} = P_t$ x (Cap + Ftg Height) x Length =	79.87 kips	Values greater than "0" if a semi-integral abutment is used
M_{E1} = 1/2 x (Cap + Ftg Height) x P_{E1} =	239.60 kip-ft	
P_{E2} = 1/2 x (P_b-P_t) x (Backwall + Cap +Ftg Ht.) x Length =	27.34 kips	
M_{E2} = 1/3 x (Backwall + Cap +Ftg Ht.) x P_{E2} =	41.77 kip-ft	
P_{ETotal} =	107.21 kips	
M_{ETotal} =	281.37 kip-ft	

Calculations For	Innerbelt Bridge No. 6	Job No.	49633	Sheet No.
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TITLE: Forward Abutment Pile Design

NOTE: Input data is denoted by shading. All other values are calculations performed by the spreadsheet.

REFERENCE: 2007 AASHTO LRFD, 4th Edition, with Interims Through 2009

LL Surcharge:

Is there an approach slab, Yes or No?	No	If "Yes", then values in final condition design are zeroed out.
P_S = 2ft x EFP x Height x Length =	23.86 kips	**Say "No" since ODOT BDM S3.11.6.5 states that no reduction should be applied even with the presence of an approach slab.**
M_E = 1/2 x Height x P_S =	71.59 kip-ft	

Temperature:

Temperature Rise/Fall, T =	60 degrees	Determined using a 60° median temperature and Table 3.12.2.1-1
Coefficient of Expansion and Contraction, C_{EC} =	0.000006	per degree Fahrenheit (AASHTO 5.4.2.2 for Concrete & 6.4.1 for Steel)
Length Contributing to Temperature Movement, L_T =	0.000 ft	**Do not consider temperature movement for semi-integral abutment per ODOT BDM 306.3.1**
Δ_T = T x C_{EC} x L_T x 12 in/ft =	0.000 in	
Bearing Pad Width, P_W =	20.000 in	
Bearing Pad Length, P_L =	12.000 in	
Bearing Pad Thickness, P_T =	1.844 in	Elastomer Thickness Only
Bearing Pad Shear Modulus, G =	130 psi	Use highest value within range for Durometer Hardness 50 to maximize load
Number of Pads per Beam, n_P =	1	
Distance Top of Cap to Bottom of Footing, D_{CF} =	6.000 ft	
P_{Temp} = Δ_T x P_W x P_L x G x n_P x No. Beams / P_T =	0.00 kips	
M_T = P_{Temp} x D_{CF} =	0.00 kip-ft	

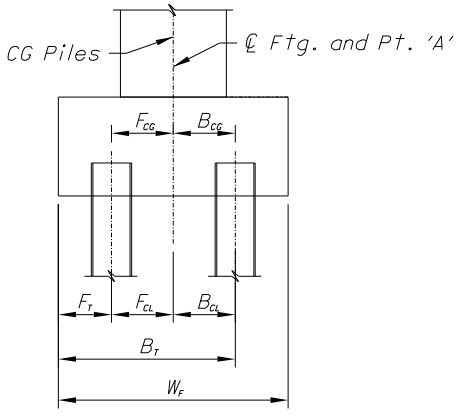
Earthquake:

Note: Per AASHTO 3.10.9.2 take 25% of the vertical reaction (100% to be applied in longitudinal direction only).

	Vertical Reaction	Lateral Load	Arm	Moment
DL Superstructure, DC	1084.60 kips	271.15 kips	3.000 ft	813.45 kip-ft
End Diaphragm	187.77 kips	46.94 kips	3.000 ft	140.83 kip-ft
Superstructure LL (50% During Event)	360.00 kips	90.00 kips	3.000 ft	270.00 kip-ft
		408.09 kips		1224.28 kip-ft

Footing and Pile Input

Footing Width, W_F =	6.500 ft	
Distance from Front Pile to Toe, F_T =	1.50 ft	
Distance from Back Pile to Toe, B_T =	5.000 ft	
Distance from Front Pile to CL Footing, F_{CL} =	1.750 ft	
Distance from Back Pile to CL Footing, B_{CL} =	1.750 ft	
Number of Front Piles, n_F =	10	
Number of Back Piles, n_B =	10	
Pile Size or Diameter =	15.7 in	
Maximum Pile Spacing =	8.000 ft	per ODOT BDM 303.4.2.2
Anticipated Front Row Pile Spacing =	6.593 ft	**OK!**
Anticipated Back Row Pile Spacing =	6.593 ft	**OK!**
Factored Nominal Pile Axial Resistance =	234.00 kips	← 360 kip, HP16x88 Steel H-Piles @ 88' depth per pile curves, Φ = 0.65
Distance from 'A' to CG Piles, A_{CG} =	0.000 ft	
Distance from Front Pile to CG, F_{CG} =	1.750 ft	
Distance from Back Pile to CG, B_{CG} =	1.750 ft	
nd^2 = n_F x F_{CG}^2 + n_B x B_{CG}^2 =	61.250	
Are the front row of piles battered, Yes or No?	No	
Downdrag Force =	0.00 kips	None anticipated per Geotech



Calculations For	Innerbelt Bridge No. 6	Job No.	49633	Sheet No.
Made by	AKS	Date	4/20/2011	1/10/2012
Checked by	DBT	Date	4/22/2011	1/16/2012
Backchecked by	AKS	Date	4/25/2011	1/16/2012

TITLE: Forward Abutment Pile Design

NOTE: Input data is denoted by shading. All other values are calculations performed by the spreadsheet.

REFERENCE: 2007 AASHTO LRFD, 4th Edition, with Interims Through 2009

Strength I - Final Condition Case

Load Factors (AASHTO Table 3.4.1-1 & 3.4.1-2)

For DC Dead Loads, γ_{pDC} =	1.25
For FWS Load (DW Max), γ_{pDW} =	1.50
For Live Loads, γ_{LL} =	1.75
For Downdrag, γ_{pDD} =	1.40
For Earth Pressure, γ_{pEH} =	1.50
For Earth LL Surcharge, γ_{pES} =	1.50
For Uniform Temperature, γ_{TU} =	1.20

Summary of Factored Loads

	Vertical Forces			Horizontal Forces	
	P_V	M_A		P_H	M_A
DL Abutment	728.48 kips	659.65 kip-ft	Earth Pressure	160.81 kips	422.06 kip-ft
DL Superstructure, DC	1590.46 kips	0.00 kip-ft	LL Surcharge	41.76 kips	125.27 kip-ft
DL Super. FWS, DW Max	363.75 kips	0.00 kip-ft	Temperature	0.00 kips	0.00 kip-ft
Superstructure LL	1260.00 kips	0.00 kip-ft			
Pedestrian LL	0.00 kips	0.00 kip-ft			
Downdrag	0.00 kips	0.00 kip-ft			

Total Vertical Forces, P_V =	3942.70 kips	Does not include downdrag		
Total Horizontal Forces, P_H =	202.57 kips			
Total Moments, M_A =	1206.98 kip-ft			
$M_{CG} = M_A - P_V \times A_{CG}$ =	1206.98 kip-ft			
$F_{VFront} = P_V / (n_F + n_B) + M_{CG} \times F_{CG} / nd^2$ =	231.62 kips			
$F_{HFront} = F_{VFront} \times {}^4/_{12}$ Batter (or = 0 without batter) =	0.00 kips	= 0 kips if no batter on front row of piles		
Net Horizontal = $(P_H - n_F \times F_{HFront}) / n_{Total}$ =	10.13 kips	= P_h / Total # piles if no batter on front row of piles		
$Axial_{Front} = ({}^{12.6491}/_{12}) \times F_{VFront}$ (with batter or = F_{VFront} w/out) =	231.62 kips	**OK**	with downdrag =	231.62 kips **OK**
$Axial_{Back} = P_V / n_{Total} - M_{CG} \times B_{CG} / nd^2$ =	162.65 kips	**OK**	with downdrag =	162.65 kips **OK**

Strength I - Construction Condition Case

Includes LL surcharge but does not include superstructure LL or future wearing surface.

Load Factors (AASHTO Table 3.4.1-1 & 3.4.1-2)

For DC Dead Loads, γ_{pDC} =	1.25
For FWS Load (DW Max), γ_{pDW} =	1.50
For Live Loads, γ_{LL} =	1.75
For Downdrag, γ_{pDD} =	1.40
For Earth Pressure, γ_{pEH} =	1.50
For Earth LL Surcharge, γ_{pES} =	1.50
For Uniform Temperature, γ_{TU} =	1.20

Summary of Factored Loads

	Vertical Forces			Horizontal Forces	
	P_V	M_A		P_H	M_A
DL Abutment	728.48 kips	659.65 kip-ft	Earth Pressure	160.81 kips	422.06 kip-ft
DL Superstructure, DC	1590.46 kips	0.00 kip-ft	LL Surcharge	41.76 kips	125.27 kip-ft
DL Super. FWS, DW Max	0.00 kips	0.00 kip-ft	Temperature	0.00 kips	0.00 kip-ft
Superstructure LL	0.00 kips	0.00 kip-ft			
Pedestrian LL	0.00 kips	0.00 kip-ft			
Downdrag	0.00 kips	0.00 kip-ft			

Total Vertical Forces, P_V =	2318.95 kips	Does not include downdrag		
Total Horizontal Forces, P_H =	202.57 kips			
Total Moments, M_A =	1206.98 kip-ft			
$M_{CG} = M_A - P_V \times A_{CG}$ =	1206.98 kip-ft			
$F_{VFront} = P_V / (n_F + n_B) + M_{CG} \times F_{CG} / nd^2$ =	150.43 kips			
$F_{HFront} = F_{VFront} \times {}^4/_{12}$ Batter (or = 0 without batter) =	0.00 kips	= 0 kips if no batter on front row of piles		
Net Horizontal = $(P_H - n_F \times F_{HFront}) / n_{Total}$ =	10.13 kips	= P_h / Total # piles if no batter on front row of piles		
$Axial_{Front} = ({}^{12.6491}/_{12}) \times F_{VFront}$ (with batter or = F_{VFront} w/out) =	150.43 kips	**OK**	with downdrag =	150.43 kips **OK**
$Axial_{Back} = P_V / n_{Total} - M_{CG} \times B_{CG} / nd^2$ =	81.46 kips	**OK**	with downdrag =	81.46 kips **OK**

Calculations For	Innerbelt Bridge No. 6	Job No.	49633	Sheet No.
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Checked by	DBT	Date	4/22/2011	1/16/2012
Backchecked by	AKS	Date	4/25/2011	1/16/2012

TITLE: Forward Abutment Pile Design

NOTE: Input data is denoted by shading. All other values are calculations performed by the spreadsheet.

REFERENCE: 2007 AASHTO LRFD, 4th Edition, with Interims Through 2009

Exreme Event I

Load Factors (AASHTO Table 3.4.1-1 & 3.4.1-2)

For DC Dead Loads, γ_{pDC} =	1.25
For FWS Load (DW Max), γ_{pDW} =	0.00
For Live Loads, γ_{LL} =	1.00
For Downdrag, γ_{pDD} =	1.40
For Earth Pressure, γ_{EH} =	0.00
For Earthquake, γ_{EQ} =	1.00

Summary of Factored Loads

	Vertical Forces			Horizontal Forces	
	P_V	M_A		P_H	M_A
DL Abutment	728.48 kips	659.65 kip-ft	Earth Pressure	0.00 kips	0.00 kip-ft
DL Superstructure, DC	1590.46 kips	0.00 kip-ft	LL Surcharge	23.86 kips	71.59 kip-ft
DL Super FWS, DW Max	0.00 kips	0.00 kip-ft	Earthquake	408.09 kips	1224.28 kip-ft
Superstructure Live Load	0.00 kips	0.00 kip-ft			
Downdrag	0.00 kips	0.00 kip-ft			

Total Vertical Forces, P_V =	2318.95 kips			
Total Horizontal Forces, P_H =	431.95 kips			
Total Moments, M_A =	1955.51 kip-ft			
$M_{CG} = M_A - P_V \times A_{CG}$ =	1955.51 kip-ft			
$F_{VFront} = P_V / (n_F + n_B) + M_{CG} \times F_{CG} / nd^2$ =	171.82 kips			
$F_{HFront} = F_{VFront} \times {}^{4}/_{12}$ Batter (or = 0 without batter) =	0.00 kips	= 0 kips if no batter on front row of piles		
Net Horizontal = $(P_H - n_F \times F_{HFront}) / n_{Total}$ =	21.60 kips	= P_h / Total # piles if no batter on front row of piles		
$Axial_{Front} = ({}^{12.6491}/_{12}) \times F_{VFront}$ (with batter or = F_{VFront} w/out) =	171.82 kips	**OK**	with downdrag =	171.82 kips **OK**
$Axial_{Back} = P_V / n_{Total} - M_{CG} \times B_{CG} / nd^2$ =	60.08 kips	**OK**	with downdrag =	60.08 kips **OK**

Calculations For	Innerbelt Bridge No. 6	Job No.	49633	Sheet No.
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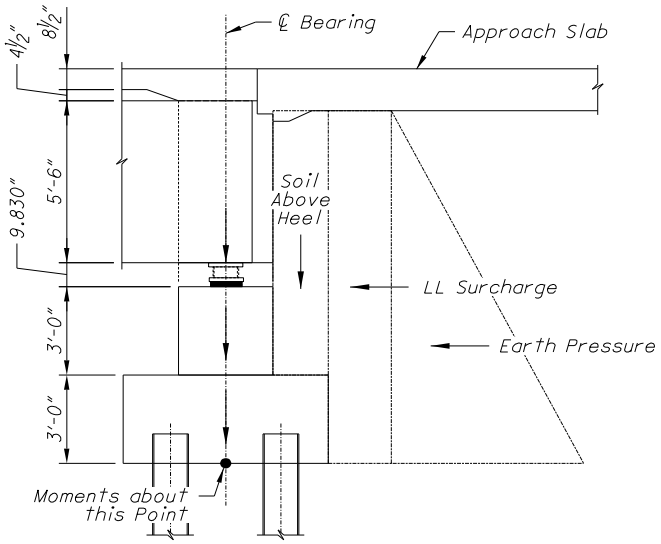
Backchecked by	AKS	Date	4/25/2011	1/16/2012

TITLE: Forward Abutment Pile Design - Section over Existing 66" Sewer

NOTE: Input data is denoted by shading. All other values are calculations performed by the spreadsheet.

REFERENCE: 2007 AASHTO LRFD, 4th Edition, with Interims Through 2009



Vertical Loads & Moments

Item	Height (ft)	Width (ft)	Length (ft)	Weight (kips)	Moment Arm (ft)	Moment About CL Ftg (k-ft)	Note: Moments are taken about the CL of footing and if arm is to the left of CL footing, per the figures above, input as a negative value.
Backwall	0.000	0.000	0.000	0.00	0.000	0.00	
Cap	3.000	3.000	27.510	37.14	0.000	0.00	
Footing	3.000	6.500	27.510	80.47	0.000	0.00	
Soil on Heel	8.986	1.750	27.510	51.91	2.375	123.29	
Approach Slab	1.417	30.000	27.510	87.69	1.250	109.61	←Assume 1/2 is distributed to abut.
Wings	-	0.000	-	0.00	0.000	0.00	MSE Wall in lieu of Wingwall
Substructure DL				257.21		232.90	

	Number of Beams	3					
	Ext. Beam Rxn (kips)	Int. Beam Rxn (kips)					
DL Superstructure, DC	154.3	155.2		463.80	0.000	0.00	Rxn = Self Wt + DL Prec DC + Deck+Haunch + Diaphr
DL Super. FWS, DW	32.50	35.50		100.50	0.000	0.00	Rxn = DL Comp DW
Superstructure LL (no Impact)	96.09	105.57		297.74	0.000	0.00	Rxn = LL+I / 1.15
Pedestrian LL	0.00	0.00		0.00	0.000	0.00	
	Height (ft)	Width (ft)	Length (ft)				
End Diaphragm	6.694	3.000	27.510	82.87	0.000	0.00	Height = haunch + beam + bearing

Horizontal Loads & Moments

Earth Pressure:

Abutment Type =	Semi-Integral	
Equivalent Fluid Pressure, EFP =	31.90 pcf	
Length =	27.51 ft	
Pressure @ Top of Non-Int. Backwall or @ Top of Semi-Int. Cap, P_t =	213.55 psf	
Pressure @ Bottom of Cap or Footing, P_b =	404.95 psf	
$P_{E1} = P_t$ x (Cap + Ftg Height) x Length =	35.25 kips	Values greater than "0" if a semi-integral abutment
M_{E1} = 1/2 x (Cap + Ftg Height) x P_{E1} =	105.75 kip-ft	is used
P_{E2} = 1/2 x (P_b-P_t) x (Backwall + Cap +Ftg Ht.) x Length =	12.07 kips	
M_{E2} = 1/3 x (Backwall + Cap +Ftg Ht.) x P_{E2} =	18.44 kip-ft	
P_{ETotal} =	47.32 kips	
M_{ETotal} =	124.18 kip-ft	

Calculations For	Innerbelt Bridge No. 6	Job No.	49633	Sheet No.
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TITLE: Forward Abutment Pile Design - Section over Existing 66" Sewer

NOTE: Input data is denoted by shading. All other values are calculations performed by the spreadsheet.

REFERENCE: 2007 AASHTO LRFD, 4th Edition, with Interims Through 2009

LL Surcharge:

Is there an approach slab, Yes or No?	No	If "Yes", then values in final condition design are zeroed out.
P_S = 2ft x EFP x Height x Length =	10.53 kips	Say "No" since ODOT BDM S3.11.6.5 states that no reduction should be applied even with the presence of an approach slab.
M_E = 1/2 x Height x P_S =	31.59 kip-ft	

Temperature:

Temperature Rise/Fall, T =	60 degrees	Determined using a 60° median temperature and Table 3.12.2.1-1
Coefficient of Expansion and Contraction, C_{EC} =	0.000006	per degree Fahrenheit (AASHTO 5.4.2.2 for Concrete & 6.4.1 for Steel)
Length Contributing to Temperature Movement, L_T =	0.000 ft	Do not consider temperature movement for semi-integral abutment per ODOT BDM 306.3.1
Δ_T = T x C_{EC} x L_T x 12 in/ft =	0.000 in	
Bearing Pad Width, P_W =	20.000 in	
Bearing Pad Length, P_L =	12.000 in	
Bearing Pad Thickness, P_T =	1.844 in	Elastomer Thickness Only
Bearing Pad Shear Modulus, G =	130 psi	Use highest value within range for Durometer Hardness 50 to maximize load
Number of Pads per Beam, n_P =	1	
Distance Top of Cap to Bottom of Footing, D_{CF} =	6.000 ft	
P_{Temp} = Δ_T x P_W x P_L x G x n_P x No. Beams / P_T =	0.00 kips	
M_T = P_{Temp} x D_{CF} =	0.00 kip-ft	

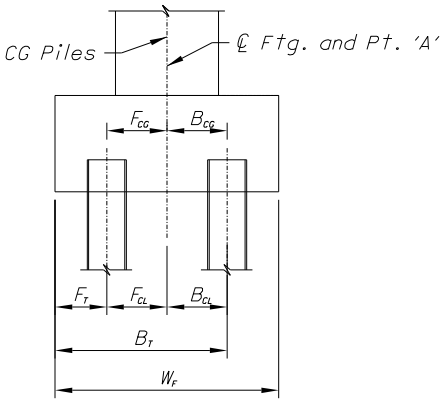
Earthquake:

Note: Per AASHTO 3.10.9.2 take 25% of the vertical reaction (100% to be applied in longitudinal direction only).

	Vertical Reaction	Lateral Load	Arm	Moment
DL Superstructure, DC	463.80 kips	115.95 kips	3.000 ft	347.85 kip-ft
End Diaphragm	82.87 kips	20.72 kips	3.000 ft	62.15 kip-ft
Superstructure LL (50% During Event)	148.87 kips	37.22 kips	3.000 ft	111.65 kip-ft
		173.89 kips		521.66 kip-ft

Footing and Pile Input

Footing Width, W_F =	6.500 ft	
Distance from Front Pile to Toe, F_T =	1.50 ft	
Distance from Back Pile to Toe, B_T =	5.000 ft	
Distance from Front Pile to CL Footing, F_{CL} =	1.750 ft	
Distance from Back Pile to CL Footing, B_{CL} =	1.750 ft	
Number of Front Piles, n_F =	4	
Number of Back Piles, n_B =	4	
Pile Size or Diameter =	15.7 in	
Maximum Pile Spacing =	8.000 ft	per ODOT BDM 303.4.2.2
Anticipated Front Row Pile Spacing =	8.170 ft	FALSE Say OK
Anticipated Back Row Pile Spacing =	8.170 ft	FALSE Say OK
Factored Nominal Pile Axial Resistance =	249.60 kips	← 384 kip, HP16x88 Steel H-Piles @ 93' depth per pile curves, Φ = 0.65
Distance from 'A' to CG Piles, A_{CG} =	0.000 ft	
Distance from Front Pile to CG, F_{CG} =	1.750 ft	
Distance from Back Pile to CG, B_{CG} =	1.750 ft	
nd^2 = n_F x F_{CG}^2 + n_B x B_{CG}^2 =	24.500	
Are the front row of piles battered, Yes or No?	No	
Downdrag Force =	0.00 kips	None anticipated per Geotech



Calculations For	Innerbelt Bridge No. 6	Job No.	49633	Sheet No.
Made by	AKS	Date	4/20/2011	1/10/2012
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Backchecked by	AKS	Date	4/25/2011	1/16/2012

TITLE: Forward Abutment Pile Design - Section over Existing 66" Sewer

NOTE: Input data is denoted by shading. All other values are calculations performed by the spreadsheet.

REFERENCE: 2007 AASHTO LRFD, 4th Edition, with Interims Through 2009

Strength I - Final Condition Case

Load Factors (AASHTO Table 3.4.1-1 & 3.4.1-2)

For DC Dead Loads, γ_{pDC} =	1.25
For FWS Load (DW Max), γ_{pDW} =	1.50
For Live Loads, γ_{LL} =	1.75
For Downdrag, γ_{pDD} =	1.40
For Earth Pressure, γ_{pEH} =	1.50
For Earth LL Surcharge, γ_{pES} =	1.50
For Uniform Temperature, γ_{TU} =	1.20

Summary of Factored Loads

	Vertical Forces			Horizontal Forces	
	P_V	M_A		P_H	M_A
DL Abutment	321.51 kips	291.13 kip-ft	Earth Pressure	70.97 kips	186.27 kip-ft
DL Superstructure, DC	683.34 kips	0.00 kip-ft	LL Surcharge	18.43 kips	55.29 kip-ft
DL Super. FWS, DW Max	150.75 kips	0.00 kip-ft	Temperature	0.00 kips	0.00 kip-ft
Superstructure LL	521.04 kips	0.00 kip-ft			
Pedestrian LL	0.00 kips	0.00 kip-ft			
Downdrag	0.00 kips	0.00 kip-ft			

Total Vertical Forces, P_V =	1676.64 kips	Does not include downdrag		
Total Horizontal Forces, P_H =	89.40 kips			
Total Moments, M_A =	532.69 kip-ft			
$M_{CG} = M_A - P_V \times A_{CG}$ =	532.69 kip-ft			
$F_{VFront} = P_V / (n_F + n_B) + M_{CG} \times F_{CG} / nd^2$ =	247.63 kips			
$F_{HFront} = F_{VFront} \times {}^4/_{12}$ Batter (or = 0 without batter) =	0.00 kips	= 0 kips if no batter on front row of piles		
Net Horizontal = $(P_H - n_F \times F_{HFront}) / n_{Total}$ =	11.18 kips	= P_h / Total # piles if no batter on front row of piles		
$Axial_{Front} = ({}^{12.6491}/_{12}) \times F_{VFront}$ (with batter or = F_{VFront} w/out) =	247.63 kips	**OK**	with downdrag = 247.63 kips	**OK**
$Axial_{Back} = P_V / n_{Total} - M_{CG} \times B_{CG} / nd^2$ =	171.53 kips	**OK**	with downdrag = 171.53 kips	**OK**

Strength I - Construction Condition Case

Includes LL surcharge but does not include superstructure LL or future wearing surface.

Load Factors (AASHTO Table 3.4.1-1 & 3.4.1-2)

For DC Dead Loads, γ_{pDC} =	1.25
For FWS Load (DW Max), γ_{pDW} =	1.50
For Live Loads, γ_{LL} =	1.75
For Downdrag, γ_{pDD} =	1.40
For Earth Pressure, γ_{pEH} =	1.50
For Earth LL Surcharge, γ_{pES} =	1.50
For Uniform Temperature, γ_{TU} =	1.20

Summary of Factored Loads

	Vertical Forces			Horizontal Forces	
	P_V	M_A		P_H	M_A
DL Abutment	321.51 kips	291.13 kip-ft	Earth Pressure	70.97 kips	186.27 kip-ft
DL Superstructure, DC	683.34 kips	0.00 kip-ft	LL Surcharge	18.43 kips	55.29 kip-ft
DL Super. FWS, DW Max	0.00 kips	0.00 kip-ft	Temperature	0.00 kips	0.00 kip-ft
Superstructure LL	0.00 kips	0.00 kip-ft			
Pedestrian LL	0.00 kips	0.00 kip-ft			
Downdrag	0.00 kips	0.00 kip-ft			

Total Vertical Forces, P_V =	1004.85 kips	Does not include downdrag		
Total Horizontal Forces, P_H =	89.40 kips			
Total Moments, M_A =	532.69 kip-ft			
$M_{CG} = M_A - P_V \times A_{CG}$ =	532.69 kip-ft			
$F_{VFront} = P_V / (n_F + n_B) + M_{CG} \times F_{CG} / nd^2$ =	163.66 kips			
$F_{HFront} = F_{VFront} \times {}^4/_{12}$ Batter (or = 0 without batter) =	0.00 kips	= 0 kips if no batter on front row of piles		
Net Horizontal = $(P_H - n_F \times F_{HFront}) / n_{Total}$ =	11.18 kips	= P_h / Total # piles if no batter on front row of piles		
$Axial_{Front} = ({}^{12.6491}/_{12}) \times F_{VFront}$ (with batter or = F_{VFront} w/out) =	163.66 kips	**OK**	with downdrag = 163.66 kips	**OK**
$Axial_{Back} = P_V / n_{Total} - M_{CG} \times B_{CG} / nd^2$ =	87.56 kips	**OK**	with downdrag = 87.56 kips	**OK**

Calculations For	Innerbelt Bridge No. 6	Job No.	49633	Sheet No.
Made by	AKS	Date	4/20/2011	1/10/2012
Checked by	DBT	Date	4/22/2011	1/16/2012
Backchecked by	AKS	Date	4/25/2011	1/16/2012

TITLE: Forward Abutment Pile Design - Section over Existing 66" Sewer

NOTE: Input data is denoted by shading. All other values are calculations performed by the spreadsheet.

REFERENCE: 2007 AASHTO LRFD, 4th Edition, with Interims Through 2009

Exreme Event I

Load Factors (AASHTO Table 3.4.1-1 & 3.4.1-2)

For DC Dead Loads, γ_{pDC} =	1.25
For FWS Load (DW Max), γ_{pDW} =	0.00
For Live Loads, γ_{LL} =	1.00
For Downdrag, γ_{pDD} =	1.40
For Earth Pressure, γ_{EH} =	0.00
For Earthquake, γ_{EQ} =	1.00

Summary of Factored Loads

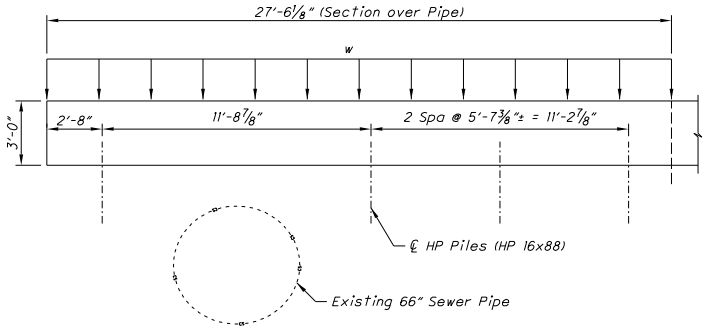
	Vertical Forces			Horizontal Forces	
	P_V	M_A		P_H	M_A
DL Abutment	321.51 kips	291.13 kip-ft	Earth Pressure	0.00 kips	0.00 kip-ft
DL Superstructure, DC	683.34 kips	0.00 kip-ft	LL Surcharge	10.53 kips	31.59 kip-ft
DL Super FWS, DW Max	0.00 kips	0.00 kip-ft	Earthquake	173.89 kips	521.66 kip-ft
Superstructure Live Load	0.00 kips	0.00 kip-ft			
Downdrag	0.00 kips	0.00 kip-ft			

Total Vertical Forces, P_V =	1004.85 kips			
Total Horizontal Forces, P_H =	184.42 kips			
Total Moments, M_A =	844.38 kip-ft			
$M_{CG} = M_A - P_V \times A_{CG}$ =	844.38 kip-ft			
$F_{VFront} = P_V / (n_F + n_B) + M_{CG} \times F_{CG} / nd^2$ =	185.92 kips			
$F_{HFront} = F_{VFront} \times {}^{4}/_{12}$ Batter (or = 0 without batter) =	0.00 kips	= 0 kips if no batter on front row of piles		
Net Horizontal = $(P_H - n_F \times F_{HFront}) / n_{Total}$ =	23.05 kips	= P_h / Total # piles if no batter on front row of piles		
$Axial_{Front} = ({}^{12.6491}/_{12}) \times F_{VFront}$ (with batter or = F_{VFront} w/out) =	185.92 kips	**OK**	with downdrag =	185.92 kips **OK**
$Axial_{Back} = P_V / n_{Total} - M_{CG} \times B_{CG} / nd^2$ =	65.29 kips	**OK**	with downdrag =	65.29 kips **OK**

Calculations For	Innerbelt Bridge No. 6	Job No.	49633		Sheet No.
Made by	AKS	Date	4/20/2011	7/7/2011	1/10/2012
Checked by	DBT	Date	4/22/2011		1/16/2012
Backchecked by	AKS	Date	4/25/2011		1/16/2012

HNTB

TITLE: Forward Abutment - Additional Reinforcing at Section Over Pipe



Bottom Longitudinal Steel Over Sanitary Pipe

Max. Factored Load Case =	Strength I, Final	
Max. Factored Vertical Load, w =	60.95	k/ft
Max. Pile Spa. Over Pipe =	11.74	ft/ft
Pos. Moment Between Piles, M_u^+ =	526.39	k-ft/ft (See SAP Results)

Height, h =	36.00 in		Clear Cover, C_v =	3.00 in
Width, b =	78.00 in		Stirrup Size =	None
Number of Bars in Row 1 =	5	#11 bar	Vertical Row Spacing =	0.00 in
Number of Bars in Row 2 =	0	#4 bar	f_y =	60 ksi
Number of Bars in Row 3 =	0	#4 bar	f'_c =	4 ksi
Number of Bars in Row 4 =	0	#4 bar	n =	7.56

Factored Moment, M_U = 526.4 k-ft

Minimum Reinforcement

AASHTO 5.7.3.3.2

$1.2M_{cr} = 1.2\ (0.37\ \sqrt{f'_c})\ 1/6\ b\ h^2$ = 1246.8 k-ft

Maximum Reinforcement

This criteria was eliminated in 2005 interims, see Commentary in Section 5.7.3.3.1

Moment Capacity

AASHTO 5.7.3.2.2

Design Moment, M_{Design} =	701.9 k-ft	
Area of Steel, A_s =	7.80 in^2	
Centroid of Reinforcement, C_R =	3.705 in	
Effective Depth, $d_e = h - C_v$ - Stirrup Bar - C_R =	32.295 in	
$a = (A_s f_y) / (0.85 f'_c b)$ =	1.765 in	AASHTO 5.7.2.2
$\phi M_n = 0.9 A_s f_y (d_e - a/2)$ =	1102.6 k-ft	**OK**

Use	5	#11	@	18" Max.

Calculations For	Innerbelt Bridge No. 6	Job No.	49633		Sheet No.
Made by	AKS	Date	4/20/2011	7/7/2011	1/10/2012
Checked by	DBT	Date	4/22/2011		1/16/2012
Backchecked by	AKS	Date	4/25/2011		1/16/2012

HNTB

TITLE: Forward Abutment - Additional Reinforcing at Section Over Pipe

Top Longitudinal Steel Over Piles on either side of Sanitary Pipe

Neg. Moment at Piles, M_{u-} = 584.72 k-ft/ft (See SAP Results)

Height, h =	36.00 in		Clear Cover, C_v =	3.00 in
Width, b =	78.00 in		Stirrup Size =	None
Number of Bars in Row 1 =	5	#10 bar	Vertical Row Spacing =	0.00 in
Number of Bars in Row 2 =	0	#4 bar	f_y =	60 ksi
Number of Bars in Row 3 =	0	#4 bar	f'_c =	4 ksi
Number of Bars in Row 4 =	0	#4 bar	n =	7.56

Factored Moment, M_U = 584.7 k-ft

Minimum Reinforcement

AASHTO 5.7.3.3.2

$1.2M_{cr} = 1.2\ (0.37\ \sqrt{f'_c})\ 1/6\ b\ h^2$ = 1246.8 k-ft

Maximum Reinforcement

This criteria was eliminated in 2005 interims, see Commentary in Section 5.7.3.3.1

Moment Capacity

AASHTO 5.7.3.2.2

Design Moment, M_{Design} =	779.6 k-ft	
Area of Steel, A_s =	6.35 in^2	
Centroid of Reinforcement, C_R =	3.635 in	
Effective Depth, $d_e = h - C_v$ - Stirrup Bar - C_R =	32.365 in	
$a = (A_s f_y) / (0.85 f'_c b)$ =	1.437 in	AASHTO 5.7.2.2
$\phi M_n = 0.9 A_s f_y (d_e - a/2)$ =	904.3 k-ft	**OK**

Use	5	#10	@	18" Max.

Calculations For	Innerbelt Bridge No. 6	Job No.	49633		Sheet No.
Made by	AKS	Date	4/20/2011	7/7/2011	**1/10/2012**
Checked by	DBT	Date	4/22/2011		**1/16/2012**
Backchecked by	AKS	Date	4/25/2011		**1/16/2012**

HNTB

TITLE: Forward Abutment - Additional Reinforcing at Section Over Pipe

Shear Reinforcement

AASHTO 5.8

V_u =	377.99 kips			
Stirrup Size =	#6 bar			
No. Legs =	4			
d_v = Greater of $0.9d_e$, 0.72 h, or $(d_e - a/2)$ =	31.413 in	Eq'n (5.8.2.9-1)		
β =	2.0	AASHTO 5.8.3.4.1		
$\phi V_c = 0.9\ (0.0316)\ \beta\ \sqrt{f'_c}\ b\ d_v$ =	278.73 kips	Eq'n (5.8.3.3-3)		
$V_{s,Req'd}$ =	110.29 kips			
v_c =	0.17 ksi			
s =	21.42 in	Eq'ns (5.8.2.5-1) & (5.8.3.3-4)	Req'd Spacing =	21.42 in
s_{max} =	24.00 in	AASHTO 5.8.2.7	Actual Spacing =	9.00 in **OK**

AASHTO 5.8.3.5

T =	459.13 kips	Eq'n (5.8.3.5-1)
T_{actual} =	468.00 kips	**OK**

Calculations For	Innerbelt Bridge No. 6	Job No.	49633	Sheet No.
Made by	AKS	Date	4/20/2011	
Checked by	DBT	Date	4/22/2011	
Backchecked by	AKS	Date	4/25/2011	

TITLE: Ultimate Bearing Values

Per ODOT DBM 202.2.3.2.b

$$UltimateBearingValue, UBV = \frac{Max.FactoredPileLoad\,(tons)}{RF}$$

	Max Factored Pile Load	RF	UBV
Rear Abutment	116.9 tons	0.65	179.9 tons
Forward Abutment (Typical Length)	115.8 tons	0.65	178.2 tons
Forward Abutment (Length over Pipe)	123.8 tons	0.65	190.5 tons

Calculations For	Innerbelt Bridge No. 6	Job No.	49633	Sheet No.
Made by	AKS	Date	4/20/2011	1/10/2012
Checked by	DBT	Date	4/22/2011	1/16/2012
Backchecked by	AKS	Date	4/25/2011	1/16/2012

HNTB

TITLE: Rear Abutment Pile Bending Check

Pile Size =	HP 16x88
Pile Area, A_s =	25.8 in2
S_x =	145 in3
F_y =	50 ksi
Unbraced Length, L_b =	10.0 ft
φ_f =	1.0 AASHTO 6.5.4.2
φ_c =	0.7 AASHTO 6.5.4.2
λ =	0 AASHTO 10.7.3.13

$P_r = \varphi P_n = \varphi_c 0.66^{\lambda} F_y A_s$ = 903.0 kip/pile AASHTO 6.9.4.1

$M_{rx} = \varphi M_n = \varphi_f F_y S_x$ = 604.2 kip-ft

Downdrag =	130.00 kips
For Downdrag, γ_{pDD} =	1.40
Factored Downdrag Load =	182.00 kips

	Strength I, Final (D + LL)	Strength I, Final (D + Downdrag)	Strength I, Construction	Extreme Event I
Max. Factored Acial Pile Force, P_u =	233.9 kip/pile	348.0 kip/pile	150.9 kip/pile	173.1 kip/pile
Lateral Factored Pile Force, V_u =	9.7 kip/pile	7.9 kip/pile	10.0 kip/pile	22.0 kip/pile
Bending Moment, $M_u = V_u L_b$ =	97.1 kip-ft/pile	79.4 kip-ft/pile	100.1 kip-ft/pile	219.7 kip-ft/pile
P_u / P_r =	0.26	0.39	0.17	0.19
M_u / M_r =	0.16	0.13	0.17	0.36
Interaction Equation = $P_u/P_r + 8/9 (M_u/M_r) \le 1.0$	0.40	0.50	0.31	0.51
AASHTO 6.9.2.2-2	OK	OK	OK	OK

Calculations For	Innerbelt Bridge No. 6	Job No.	49633	Sheet No.
Made by	AKS	Date	4/20/2011	1/10/2012
Checked by	DBT	Date	4/22/2011	1/16/2012
Backchecked by	AKS	Date	4/25/2011	1/16/2012

HNTB

TITLE: Forward Abutment Pile Bending Check

Pile Size =	HP 16x88	
Pile Area, A_s =	25.8 in2	
S_x =	145 in3	
F_y =	50 ksi	
Unbraced Length, L_b =	10.0 ft	
φ_f =	1.0	AASHTO 6.5.4.2
φ_c =	0.7	AASHTO 6.5.4.2
λ =	0	AASHTO 10.7.3.13

$P_r = \varphi P_n = \varphi_c 0.66^{\lambda} F_y A_s$ = 903.0 kip/pile AASHTO 6.9.4.1

$M_{rx} = \varphi M_n = \varphi_f F_y S_x$ = 604.2 kip-ft

Downdrag =	130.00 kips
For Downdrag, γ_{pDD} =	1.40
Factored Downdrag Load =	182.00 kips

	Strength I, Final (D + LL)	Strength I, Final (D + Downdrag)	Strength I, Construction	Extreme Event I
Max. Factored Acial Pile Force, P_u =	231.6 kip/pile	347.0 kip/pile	150.4 kip/pile	171.8 kip/pile
Lateral Factored Pile Force, V_u =	10.1 kip/pile	8.0 kip/pile	10.1 kip/pile	21.6 kip/pile
Bending Moment, $M_u = V_u L_b$ =	101.3 kip-ft/pile	80.4 kip-ft/pile	101.3 kip-ft/pile	216.0 kip-ft/pile
P_u / P_r =	0.26	0.38	0.17	0.19
M_u / M_r =	0.17	0.13	0.17	0.36
Interaction Equation = $P_u/P_r + 8/9 (M_u/M_r) \le 1.0$	0.41	0.50	0.32	0.51
AASHTO 6.9.2.2-2	OK	OK	OK	OK

Calculations For	Innerbelt Bridge No. 6	Job No.	49633	Sheet No.
Made by	AKS	Date	4/20/2011	1/10/2012
Checked by	DBT	Date	4/22/2011	1/16/2012
Backchecked by	AKS	Date	4/25/2011	1/16/2012

HNTB

TITLE: Forward Abutment (over 66" Pipe) Pile Bending Check

Pile Size =	HP 16x88	
Pile Area, A_s =	25.8 in2	
S_x =	145 in3	
F_y =	50 ksi	
Unbraced Length, L_b =	10.0 ft	
φ_f =	1.0	AASHTO 6.5.4.2
φ_c =	0.7	AASHTO 6.5.4.2
λ =	0	AASHTO 10.7.3.13

$P_r = \varphi P_n = \varphi_c 0.66^{\lambda} F_y A_s$ = 903.0 kip/pile AASHTO 6.9.4.1

$M_{rx} = \varphi M_n = \varphi_f F_y S_x$ = 604.2 kip-ft

Downdrag =	130.00 kips
For Downdrag, γ_{pDD} =	1.40
Factored Downdrag Load =	182.00 kips

	Strength I, Final (D + LL)	Strength I, Final (D + Downdrag)	Strength I, Construction	Extreme Event I
Max. Factored Acial Pile Force, P_u =	247.6 kip/pile	360.6 kip/pile	163.7 kip/pile	185.9 kip/pile
Lateral Factored Pile Force, V_u =	11.2 kip/pile	8.9 kip/pile	11.2 kip/pile	23.1 kip/pile
Bending Moment, $M_u = V_u L_b$ =	111.8 kip-ft/pile	88.7 kip-ft/pile	111.8 kip-ft/pile	230.5 kip-ft/pile
P_u / P_r =	0.27	0.40	0.18	0.21
M_u / M_r =	0.18	0.15	0.18	0.38
Interaction Equation = $P_u/P_r + 8/9\ (M_u/M_r) \le 1.0$	0.44	0.53	0.35	0.55
AASHTO 6.9.2.2-2	OK	OK	OK	OK

redriven to the original cutoff or tip elevation. At each pile group, we recommend that the central piles be driven first. Subsequent piles should be installed from the center out, proceeding in a radial pattern, or from one end to another.

It is often difficult to estimate the energy delivered by diesel hammers with visual observation. The Saximeter, developed by Pile Dynamics, Inc., can be used to record hammer strokes and provide an estimate of the driving energy of diesel hammers. If the contractor selects a diesel hammer, we recommend that a Saximeter be used during pile driving.

5.4 Settlement Considerations

Walls H and I and approach embankment fills are planned near the Bridge 6 abutments. We performed analyses to estimate both the ground surface settlement due to embankment construction and the settlement of the pile groups supporting the abutments. Pertinent calculations for bridge settlement are provided in Geotechnical Calculations GC-054 and GC-054 Addendum 3. For these analyses we considered subsurface conditions interpreted from the subsurface explorations, and abutment and fill geometries based on the available design drawings. We assumed cohesionless materials would undergo elastic settlement, and that cohesive materials would undergo consolidation settlement.

Our analyses indicate that the majority of the consolidation settlement would occur in a layer of clay ranging from about elevation 570 feet to 550 feet (about 100 to 120 feet below the existing ground surface). Based on correlations using CPT sounding C-065-0-10, this clay layer is likely normally consolidated and sensitive.

5.4.1 Embankment Settlement

Settlement of the approach embankments and associated retaining walls (Walls H and I) is presented in Geotechnical Design Memoranda GD-16 and GD-18.

5.4.2 Pile Group Settlement s and Downdrag

As indicated in AASHTO (2010) Section 3.11.8, downdrag is considered to act on piles when the soil adjacent to the piles settles 0.4 inches or more relative to the piles. To evaluate downdrag potential according to this criterion, we compared settlement values along the length of the piles to settlement of the adjacent ground considering the effects of external loading (i.e., embankment fill) and pile loading.

We used the equivalent footing method described in AASHTO (2010) Section 10.7.2.3 and Hannigan and others (2006) to estimate pile group settlement, considering the effects of the approach embankments, the foundation layout, and foundation loads provided by the DBT. Pertinent calculations are provided in GC-054. Based on loads provided by the DBT, we assumed that piles would be driven about 95 feet below the existing ground surface for both the Rear and Forward Abutments. For the settlement analyses, we conservatively modeled the fill as being placed instantaneously.

Estimated settlements for the Rear and Forward Abutments after the abutments are loaded are illustrated in Figures GD-17-12 and GD-17-13, respectively. The results of our analyses of the pile groups indicate a total settlement (immediate plus primary consolidation) of approximately 2-1/4 inches at the Rear Abutment and approximately 2 inches at the Forward Abutments following abutment loading. We estimate that less than ½-inch of the total settlement would occur essentially as the abutment load is applied (elastic settlement) and that the remaining consolidation settlement would occur relatively slowly over a period of several years.

We estimate a long-term differential settlement between the approach embankment and the Rear and Forward Abutments of about 0.4 to 0.6 inch over the 30-foot-long approach slabs with the abutments settling relative to the adjacent ground. These estimated settlements are illustrated in Figure GD-17-14. Because the piles settle relative to the adjacent soils, long-term downdrag does not develop. Similarly for the monument piles, the long-term settlement between the pile tip and ground surface is less than 0.4 inches and is insufficient to induce downdrag loads per AASHTO (2010).

We understand the DBT may drive the Bridge 6 abutment piles prior to placing the approach embankment and nearby retaining wall fill. If the piles are driven before embankment construction, temporary downdrag forces would develop on the piles from immediate soil settlement caused by the embankment loading. We estimate that the temporary downdrag loads (unfactored) on the individual piles could be up to approximately 60 to 65 tons. However, as sufficient load is subsequently applied to the pile heads (e.g., the piles are re-struck), temporary downdrag loads would be eliminated. The temporary downdrag loads are eliminated because the pile moves down relative to the adjacent soil.

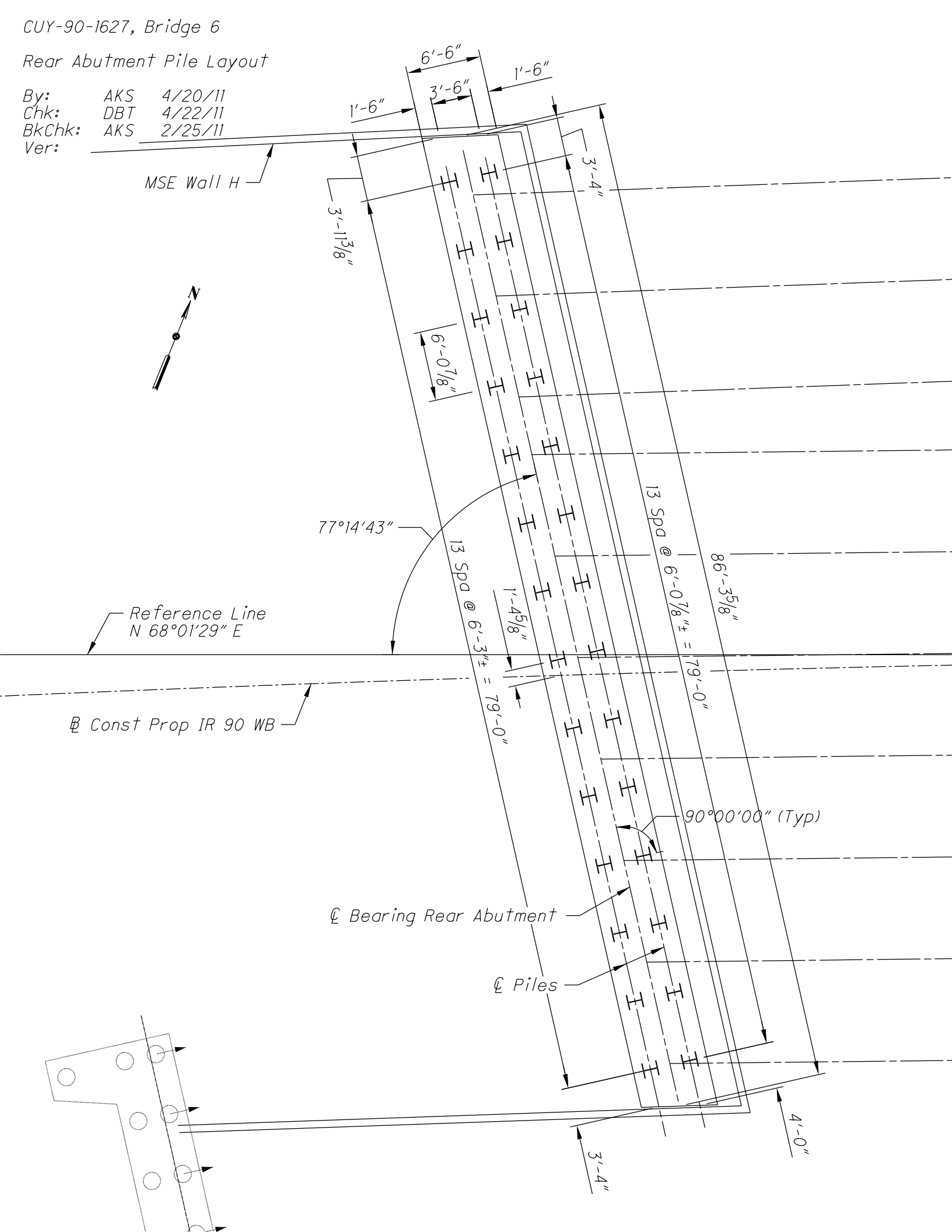
5.4.3 Settlement Monitoring and Instrumentation

We recommend that optical survey points (monuments) be provided on the exposed corner of each abutment and that measurements be taken weekly between when the abutments

CUY-90-1627, Bridge 6

Rear Abutment Pile Layout

By:	AKS	4/20/11
Chk:	DBT	4/22/11
BkChk:	AKS	2/25/11
Ver:		



CUY-90-1627, Bridge 6

Forward Abutment Pile Layout

By:	AKS	4/20/11
Chk:	DBT	4/22/11
BkChk:	AKS	4/25/11
Ver:		

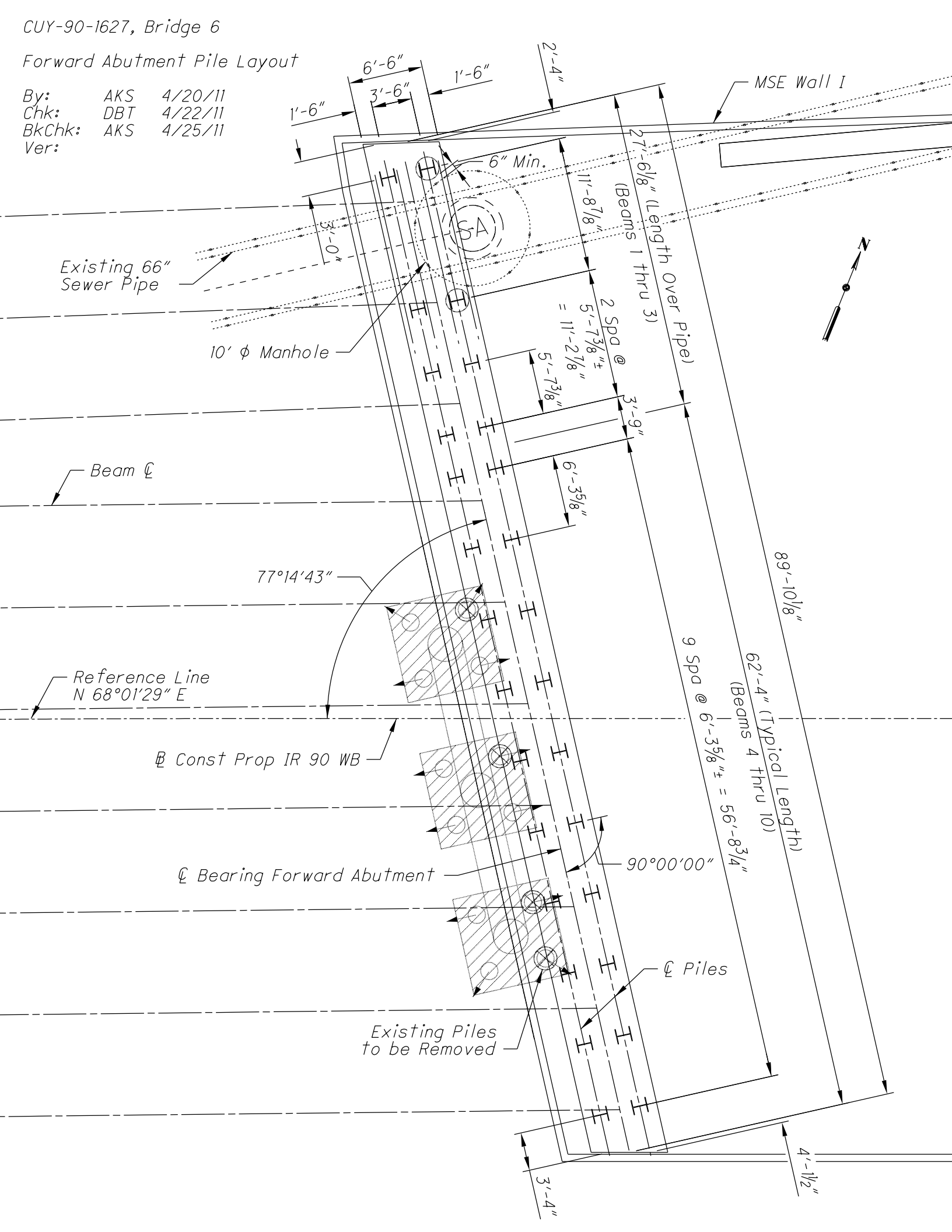


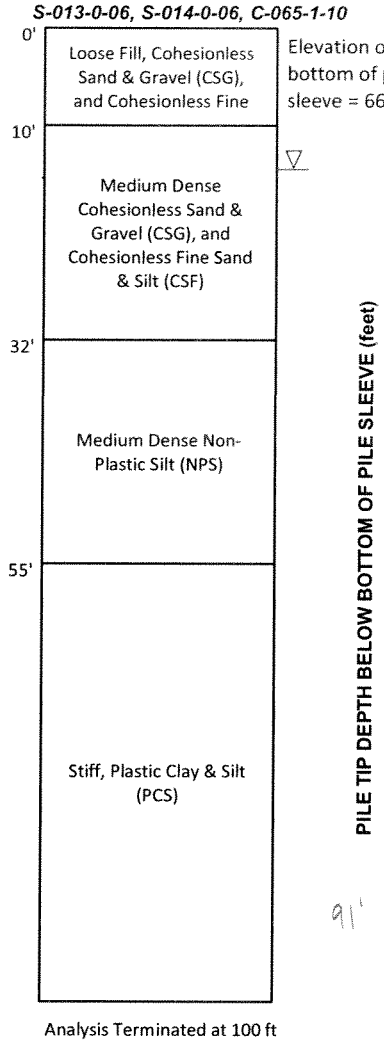
TABLE GD-2.1
RECOMMENDED DEFAULT SOIL AND ROCK PARAMETERS FOR DESIGN AND ANALYSIS

Soil or Rock Type / Engineering Soil/Rock Unit		Relative Density or Consistency	Representative N_{60}	Moist Unit Weight γ_m (lbs/ft^3)	Shear Strength Parameters	
					Static	
					Effective Stress Angle of Internal Friction ϕ' (degrees)	Undrained Shear Strength s_u (lbs/ft^2)
Fill	Common Borrow	Dense	N/A	125	34	--
	Select Granular Backfill	Dense	N/A	135	36	--
	Gravel Backfill	Dense	N/A	135	38	--
	Existing Fill	Very loose to loose	8	120	30	--
		Medium dense	15	120	32	--
		Dense to very dense	40	125	35	--
Native Soils	CSG	Very loose to loose	8	120	32	--
		Medium dense	16	125	33	--
	CSF	Very loose to loose	4	115	28	--
		Medium dense	16	120	32	--
		Dense	40	125	36	--
		Very dense	65	125	38	--
	NPS	Very loose to loose	3	115	26	--
		Medium dense	18	120	29	--
		Dense to very dense	60	125	35	--
	PCS	Medium stiff	5	115	30	650
		Stiff	12	120	30	1600
		Very stiff	22	125	30	3000
		Hard	75	130	30	5000
	OCS	Very soft	2	105	26	250
		Soft to medium stiff	4	110	26	500
	VSC	Very soft to soft	<2	110	28	250
Rock	RSH	Hard	>50/2"	140	--	q_u = 150 ksf

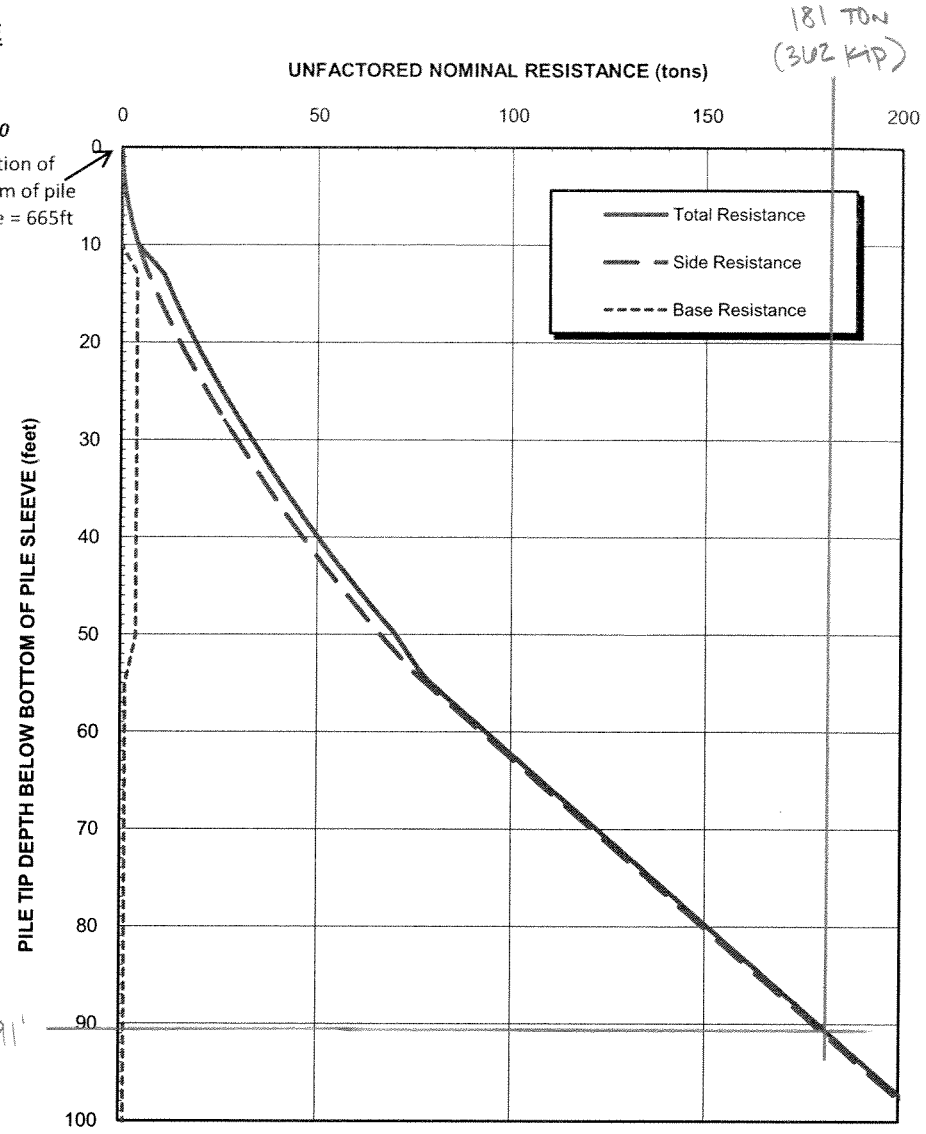
ASSUMED SUBSURFACE PROFILE

Based on Nearby Explorations:

S-013-0-06, S-014-0-06, C-065-1-10



Analysis Terminated at 100 ft



NOTES:

1. The analyses were performed based on guidelines included in the AASHTO LRFD Bridge Design Specifications (Fifth Ed., 2010), ODOT Bridge Design Manual (2007 w/ updates through 4/16/10) and our experience. The analyses are based on a single pile and do not consider group action of closely spaced piles (closer than 2.5 widths, center to center). Based on the Bridge 6 Plans, the axial resistance of the pile group may be determined by summing the resistances of the individual piles. Group reduction factors are not required.
2. Total pile capacity is a summation of its side and base resistances. Unfactored nominal resistances shown on plots above are to be multiplied by the appropriate resistance factors (RFs). Per the ODOT Bridge Design Manual, a minimum of two dynamic tests are required per each pile size at each structure, and one static load test is required when total pile length for individual structure exceeds 10,000 ft (one additional static test required for every additional 10,000 ft of pile). Assuming that piles are installed according to CMS 507 and CMS 523, an RF of 0.70 may be used. Alternatively, if dynamic testing is performed in accordance with Section 10.5.5.2.3 of LRFD Specifications, Strength Limit State RF = 0.65 for side and base resistance is appropriate. Higher RFs may be used with static load testing (See Section 10.5.5.2.3 of LRFD Specifications). Without dynamic or static testing, an RF of 0.45 should be used for side and base resistance at the Strength Limit State. An RF of 0.35 should be used for uplift resistance at the Strength Limit State. For the Extreme Event Limit State, an RF of 1.0 should be used for base and side resistance, and an RF of 0.8 should be used for uplift resistance.
3. Analysis does not consider any sesimic strength reduction.

I-90 Innerbelt CCG1
HNTB/Walsh Construction
Cleveland, Ohio

ESTIMATED AXIAL PILE RESISTANCE
HP 16x88 H-PILE
BRIDGE 6 - REAR ABUTMENT

June 2011 21-1-21361-300

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. GD-17-3

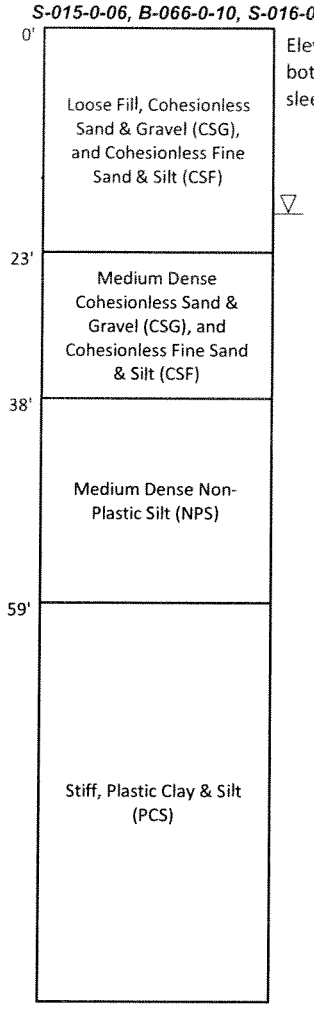
180 TON (360 KIP)

OVER PIPE 192 TON (384 KIP)

ASSUMED SUBSURFACE PROFILE

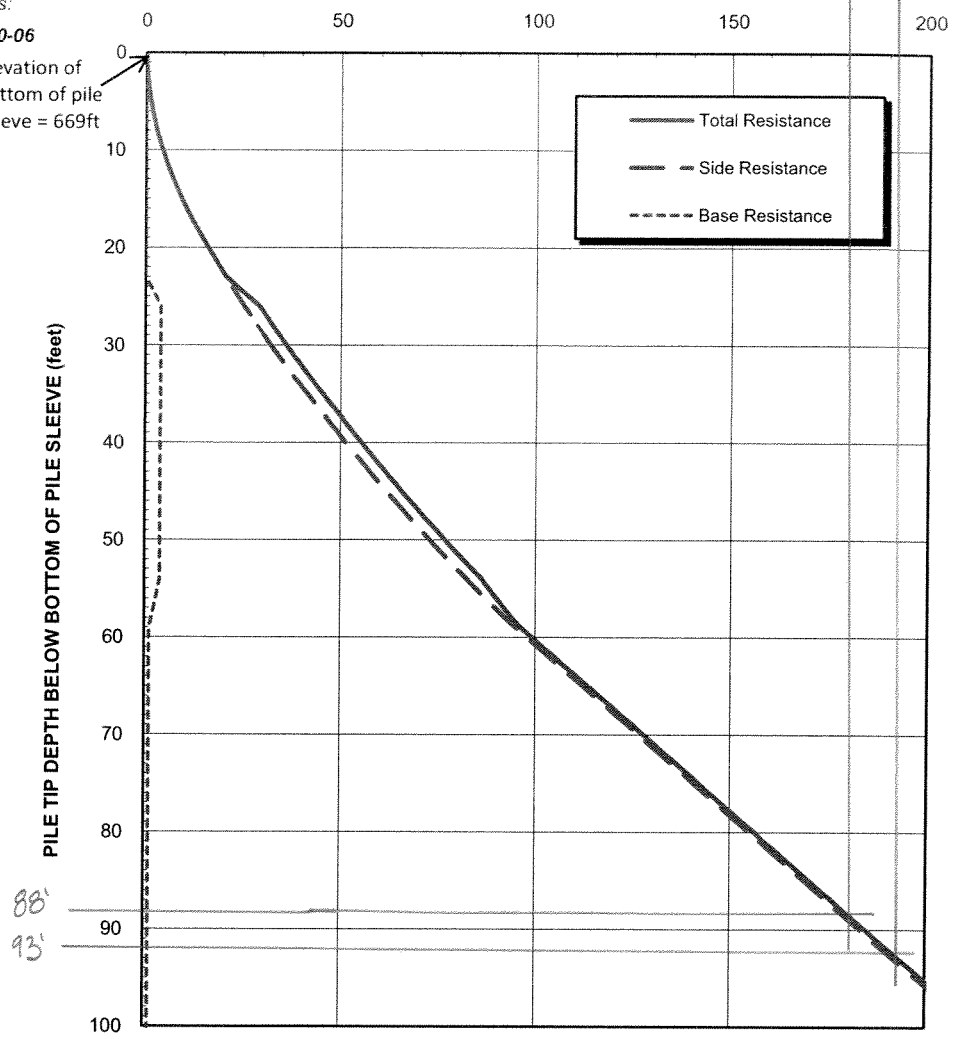
Based on Nearby Explorations:

S-015-0-06, B-066-0-10, S-016-0-06



Elevation of bottom of pile sleeve = 669ft

Analysis Terminated at 100 ft



NOTES:

1. The analyses were performed based on guidelines included in the AASHTO LRFD Bridge Design Specifications (Fifth Ed., 2010), ODOT Bridge Design Manual (2007 w/ updates through 4/16/10) and our experience. The analyses are based on a single pile and do not consider group action of closely spaced piles (closer than 2.5 widths, center to center). Based on the Bridge 6 Plans, the axial resistance of the pile group may be determined by summing the resistances of the individual piles. Group reduction factors are not required.
2. Total pile capacity is a summation of its side and base resistances. Unfactored nominal resistances shown on plots above are to be multiplied by the appropriate resistance factors (RFs). Per the ODOT Bridge Design Manual, a minimum of two dynamic tests are required per each pile size at each structure, and one static load test is required when total pile length for individual structure exceeds 10,000 ft (one additional static test required for every additional 10,000 ft of pile). Assuming that piles are installed according to CMS 507 and CMS 523, an RF of 0.70 may be used. Alternatively, if dynamic testing is performed in accordance with Section 10.5.5.2.3 of LRFD Specifications, Strength Limit State RF = 0.65 for side and base resistance is appropriate. Higher RFs may be used with static load testing (See Section 10.5.5.2.3 of LRFD Specifications). Without dynamic or static testing, an RF of 0.45 should be used for side and base resistance at the Strength Limit State. An RF of 0.35 should be used for uplift resistance at the Strength Limit State. For the Extreme Event Limit State, an RF of 1.0 should be used for base and side resistance, and an RF of 0.8 should be used for uplift resistance.
3. Analysis does not consider any sesimic strength reduction.

I-90 Innerbelt CCG1
HNTB/Walsh Construction
Cleveland, Ohio

ESTIMATED AXIAL PILE RESISTANCE
HP 16x88 H-PILE
BRIDGE 6 - FORWARD ABUTMENT

June 2011 21-1-21361-300

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. GD-17-4

Rel. Humid.(RH)	75.0	%
Es	28500.0	ksi
Eci	5250	ksi

AASHTO LOSSES

Elastic Shortening 22.18 ksi (Eq 5.9.5.2.3a-1), (fcgp= 4.086 ksi)

	Elastic Gains	Gains		Adjustment	
due	to Precast Loads	-6.97	ksi	0.81	ksi
due	to Composite Loads	-3.06	ksi	0.35	ksi
due	to Live Loads	-6.18	ksi	0.89	ksi

Time Dependent Losses (Approximate Method (Art.5.9.5.3))

		Initial		Final		
Steel relaxation	0.00	ksi		2.40	ksi	(Eq 5.9.5.3-1)
Concrete shrinkage	0.00	ksi		6.71	ksi	(Eq 5.9.5.3-1)
Concrete creep	0.00	ksi		15.22	ksi	(Eq 5.9.5.3-1)
Sub-total	22.18	ksi	(10.95 %)	10.17	ksi	(5.02 %)
Total Prestress Losses				32.34	ksi	(15.97 %)

Prestressing Stress Limit Check (Table 5.9.3.1)
initial fpi = 202.5 ksi < 0.75 fpu, OK
initial fpe = 170.2 ksi < 0.80 fpy, OK

SHEAR/MOMENT ENVELOPE (&REACTIONS)

SHEAR AND MOMENT ENVELOPE : Span : 1, Beam : 1, SERVICE I
Shears: kips, Moments: kft

		Bearing	Trans	H/2	0.10L	0.20L	0.30L	0.40L	Midspan
Location,	ft	0.00	2.17	3.06	13.14	27.11	41.09	55.06	69.04
Self wt. :	M	0.0	148.3	208.5	827.6	1516.7	2009.0	2304.3	2402.8
(Max)	V	69.6	67.4	66.5	56.4	42.3	28.2	14.1	0.0
DL-Prec. :	M	0.0	49.7	69.9	277.3	508.3	673.2	772.2	805.2
DC(Max)	V	23.3	22.6	22.3	18.9	14.2	9.4	4.7	0.0
DL-Prec. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DW(Max)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Deck + :	M	0.0	111.3	156.4	621.0	1138.2	1507.6	1729.2	1803.1
Haunch (Max)	V	52.2	50.6	49.9	42.3	31.7	21.1	10.6	0.0
Diaphragm :	M	0.0	0.9	1.2	5.3	11.0	14.9	16.8	18.6
(Max)	V	0.4	0.4	0.4	0.4	0.4	0.1	0.1	0.1
DL-Comp :	M	-0.0	18.7	26.3	104.2	191.0	253.0	290.2	302.6
DC(Max)	V	8.8	8.5	8.4	7.1	5.3	3.5	1.8	0.0
DL-Comp :	M	-0.0	69.2	97.2	385.8	707.1	936.6	1074.3	1120.2
DW(Max)	V	32.5	31.4	31.0	26.3	19.7	13.1	6.6	0.0
LL + I :	M+	-0.0	228.4	320.8	1269.3	2312.5	3037.7	3466.9	3590.2
	V	110.5	108.1	107.1	95.8	80.5	29.9	8.4	28.2
LL + I :	M-	-0.0	-0.0	-0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
LL + I :	Vmx	110.5	108.1	107.1	95.9	81.7	68.2	55.4	43.4
	M	-0.0	225.2	315.9	1224.9	2154.6	2724.5	2965.9	2916.1
Total :	M+	0.0	626.5	880.2	3490.6	6384.8	8432.0	9653.9	10042.7
	V	297.3	289.0	285.6	247.2	194.1	105.5	46.2	28.3
Total :	M-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total :	Vmx	297.3	289.0	285.6	247.2	195.3	143.8	93.2	43.6
	M	0.0	623.3	875.3	3446.2	6226.9	8118.8	9152.9	9368.6

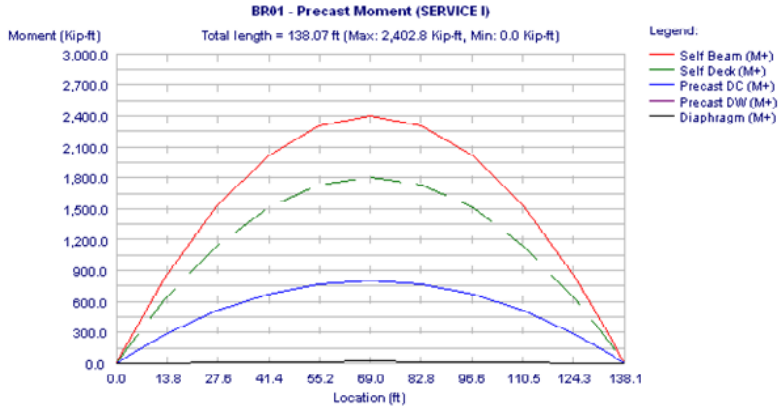
		0.60L	0.70L	0.80L	0.90L	H/2	Trans	Bearing
Location,	ft	83.01	96.98	110.96	124.93	135.01	135.91	138.07
Self wt. :	M	2304.3	2009.0	1516.7	827.6	208.5	148.3	0.0
(Max)	V	14.1	28.2	42.3	56.4	66.5	67.4	69.6
DL-Prec. :	M	772.2	673.2	508.3	277.3	69.9	49.7	0.0
DC(Max)	V	4.7	9.4	14.2	18.9	22.3	22.6	23.3
DL-Prec. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DW(Max)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Deck + :	M	1729.2	1507.6	1138.2	621.0	156.4	111.3	0.0
Haunch (Max)	V	10.6	21.1	31.7	42.3	49.9	50.6	52.2
Diaphragm :	M	16.8	14.9	11.0	5.3	1.2	0.9	-0.0
(Max)	V	0.1	0.1	0.4	0.4	0.4	0.4	0.4
DL-Comp :	M	290.2	253.0	191.0	104.2	26.3	18.7	-0.0
DC(Max)	V	1.8	3.5	5.3	7.1	8.4	8.5	8.8
DL-Comp :	M	1074.3	936.6	707.1	385.8	97.2	69.2	0.0
DW(Max)	V	6.6	13.1	19.7	26.3	31.0	31.4	32.5

		0.60L	0.70L	0.80L	0.90L	H/2	Trans	Bearing
LL + I :	M+	3466.9	3037.7	2312.5	1269.3	320.8	228.4	-0.0
	V	8.4	29.9	80.5	95.8	107.1	108.1	110.5
LL + I :	M-	0.0	0.0	0.0	0.0	-0.0	-0.0	-0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
LL + I :	Vmx	55.4	68.2	81.7	95.9	107.1	108.1	110.5
	M	2965.9	2724.5	2154.6	1224.9	315.9	225.2	0.0
Total :	M+	9653.9	8432.0	6384.8	3490.6	880.2	626.5	0.0
	V	46.2	105.5	194.1	247.2	285.6	289.0	297.3
Total :	M-	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total :	Vmx	93.2	143.8	195.3	247.2	285.6	289.0	297.3
	M	9152.9	8118.8	6226.9	3446.2	875.3	623.3	0.0

REACTIONS (kips), SERVICE I

Load Type	Left Support	Right Support
Self Wt.	69.6	69.6
Deck+Haunch	52.2	52.2
Diaphragm	0.4	0.4
DL-Prec.(DC)	23.3	23.3
DL-Prec.(DW)	0.0	0.0
DL-Comp.(DC)	93.7	93.7
DL-Comp.(DW)	347.1	347.1
Live	111.3	111.3
Pedestrian	0.0	0.0

Upward reactions are positive.
Live Load reactions are per lane with no distribution factor and no impact.
Non-composite load types are per beam.
Composite and Pedestrian load types are per total bridge width.



BR01 - Precast Moment, Span 1, Beam 1, SERVICE I

AASHTO LOSSES

Elastic Shortening 22.18 ksi (Eq 5.9.5.2.3a-1), (fcgp= 4.086 ksi)

Elastic Gains		Gains		Adjustment	
due	to Precast Loads	-6.98	ksi	0.81	ksi
due	to Composite Loads	-3.33	ksi	0.38	ksi
due	to Live Loads	-5.41	ksi	0.78	ksi

Time Dependent Losses (Approximate Method (Art.5.9.5.3))

		Initial		Final		
Steel relaxation	0.00	ksi		2.40	ksi	(Eq 5.9.5.3-1)
Concrete shrinkage	0.00	ksi		6.71	ksi	(Eq 5.9.5.3-1)
Concrete creep	0.00	ksi		15.22	ksi	(Eq 5.9.5.3-1)
Sub-total	22.18	ksi	(10.95 %)	10.58	ksi	(5.22 %)
Total Prestress Losses				32.76	ksi	(16.18 %)

Prestressing Stress Limit Check (Table 5.9.3.1)
initial fpi = 202.5 ksi < 0.75 fpu, OK
initial fpe = 169.7 ksi < 0.80 fpy, OK

SHEAR/MOMENT ENVELOPE (&REACTIONS)

SHEAR AND MOMENT ENVELOPE : Span : 1, Beam : 2, SERVICE I
Shears: kips, Moments: kft

		Bearing	Trans	H/2	0.10L	0.20L	0.30L	0.40L	Midspan
Location,	ft	0.00	2.17	3.06	13.14	27.11	41.09	55.06	69.04
Self wt. :	M	0.0	148.3	208.5	827.6	1516.7	2009.0	2304.3	2402.8
(Max)	V	69.6	67.4	66.5	56.4	42.3	28.2	14.1	0.0
DL-Prec. :	M	0.0	38.3	53.8	213.7	391.7	518.8	595.1	620.5
DC(Max)	V	18.0	17.4	17.2	14.6	10.9	7.3	3.6	0.0
DL-Prec. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DW(Max)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Deck + :	M	0.0	121.8	171.2	679.7	1245.7	1649.9	1892.5	1973.3
Haunch (Max)	V	57.2	55.4	54.6	46.3	34.7	23.1	11.6	0.0
Diaphragm :	M	0.0	1.8	2.5	10.8	22.4	30.3	34.1	38.0
(Max)	V	0.8	0.8	0.8	0.8	0.8	0.3	0.3	0.3
DL-Comp :	M	-0.0	20.4	28.7	114.1	209.0	276.9	317.6	331.1
DC(Max)	V	9.6	9.3	9.2	7.8	5.8	3.9	1.9	0.0
DL-Comp :	M	-0.0	75.7	106.4	422.3	773.9	1025.1	1175.8	1226.0
DW(Max)	V	35.5	34.4	33.9	28.8	21.6	14.4	7.2	0.0
LL + I :	M+	-0.0	201.5	283.1	1119.9	2040.3	2680.1	3058.8	3167.5
	V	121.4	118.7	117.6	105.3	88.4	32.9	9.2	30.9
LL + I :	M-	-0.0	-0.0	-0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
LL + I :	Vmx	121.4	118.7	117.6	105.3	89.7	74.9	60.8	47.7
	M	-0.0	198.7	278.7	1080.7	1901.0	2403.8	2616.8	2572.8
Total :	M+	0.0	607.9	854.2	3388.0	6199.7	8190.1	9378.2	9759.3
	V	312.1	303.5	299.9	259.8	204.6	110.0	47.9	31.2
Total :	M-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total :	Vmx	312.1	303.5	299.9	259.8	205.9	152.0	99.5	48.0
	M	0.0	605.1	849.8	3348.9	6060.4	7913.8	8936.2	9164.6

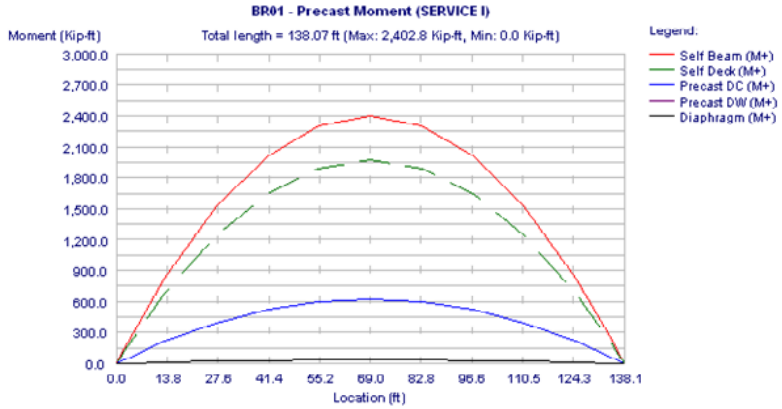
		0.60L	0.70L	0.80L	0.90L	H/2	Trans	Bearing
Location,	ft	83.01	96.98	110.96	124.93	135.01	135.91	138.07
Self wt. :	M	2304.3	2009.0	1516.7	827.6	208.5	148.3	0.0
(Max)	V	14.1	28.2	42.3	56.4	66.5	67.4	69.6
DL-Prec. :	M	595.1	518.8	391.7	213.7	53.8	38.3	0.0
DC(Max)	V	3.6	7.3	10.9	14.6	17.2	17.4	18.0
DL-Prec. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DW(Max)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Deck + :	M	1892.5	1649.9	1245.7	679.7	171.2	121.8	0.0
Haunch (Max)	V	11.6	23.1	34.7	46.3	54.6	55.4	57.2
Diaphragm :	M	34.1	30.3	22.4	10.8	2.5	1.8	-0.0
(Max)	V	0.3	0.3	0.8	0.8	0.8	0.8	0.8
DL-Comp :	M	317.6	276.9	209.0	114.1	28.7	20.4	-0.0
DC(Max)	V	1.9	3.9	5.8	7.8	9.2	9.3	9.6
DL-Comp :	M	1175.8	1025.1	773.9	422.3	106.4	75.7	0.0
DW(Max)	V	7.2	14.4	21.6	28.8	33.9	34.4	35.5

		0.60L	0.70L	0.80L	0.90L	H/2	Trans	Bearing
LL + I :	M+	3058.8	2680.1	2040.3	1119.9	283.1	201.5	-0.0
	V	9.2	32.9	88.4	105.3	117.6	118.7	121.4
LL + I :	M-	0.0	0.0	0.0	0.0	-0.0	-0.0	-0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
LL + I :	Vmx	60.8	74.9	89.7	105.3	117.6	118.7	121.4
	M	2616.8	2403.8	1901.0	1080.7	278.7	198.7	0.0
Total :	M+	9378.2	8190.1	6199.7	3388.0	854.2	607.9	0.0
	V	47.9	110.0	204.6	259.8	299.9	303.5	312.1
Total :	M-	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total :	Vmx	99.5	152.0	205.9	259.8	299.9	303.5	312.1
	M	8936.2	7913.8	6060.4	3348.9	849.8	605.1	0.0

REACTIONS (kips), SERVICE I

Load Type	Left Support	Right Support
Self Wt.	69.6	69.6
Deck+Haunch	57.2	57.2
Diaphragm	0.8	0.8
DL-Prec.(DC)	18.0	18.0
DL-Prec.(DW)	0.0	0.0
DL-Comp.(DC)	93.7	93.7
DL-Comp.(DW)	347.1	347.1
Live	111.3	111.3
Pedestrian	0.0	0.0

Upward reactions are positive.
Live Load reactions are per lane with no distribution factor and no impact.
Non-composite load types are per beam.
Composite and Pedestrian load types are per total bridge width.



BR01 - Precast Moment, Span 1, Beam 2, SERVICE I

Calculations For	Innerbelt Bridge No. 6	Job No.	49633	Sheet No.
Made by	AKS	Date	4/18/2011	1/10/2012
Checked by	DBT	Date	4/19/2011	1/16/2012
Backchecked by	AKS	Date	4/19/2011	1/16/2012

TITLE: Bearing Pad Design - Rear Abutment

ELASTOMERIC BEARING PAD DESIGN USING AASHTO LRFD SECTION 14.7.6.3 - METHOD A

	Max	Min	
DC1 Reaction	145.60 kips	145.50 kips	From Beam/Girder Output
DC2 + DW Reaction	45.10 kips	41.30 kips	
LL Reaction (No Impact)	105.57 kips	96.09 kips	
Total Unfactored DL + LL Reaction, R_{Total} =	296.27 kips	282.89 kips	

Span Length, L_{span} =	138.07 ft	
Temperature Deflection, Δ_T =	0.298 in	(Temp Rise/Fall x Coeff. Therm Exp/Contr x Length x Load Factor = 60°F x 0.000006 x (L_{span}/2) x 12 in/ft x 1.0 (γ_{TG}))
Shrinkage/Creep Deflection, Δ_C =	0.249 in	(Shrinkage/Creep Coeff. x Length = (0.0005-0.0002) x (L_{span}/2) x (12 in./ft)
Min. Req'd Elast. Thick., $t = 2(\Delta_T + \Delta_C)$ =	1.094 in	
Pad Length (along beam), L =	12.000 in	
Pad Width (across beam), W =	20.000 in	
Durometer Hardness =	50	
For Bearing Stress Check: Shear Modulus, G =	95 psi	Based on Durometer Hardness, See Table 14.7.6.2-1
For Rotation Check: Shear Modulus, G =	130 psi	
Bearing Area, A =	240.00 in^2	
Bearing Perimeter =	64.000 in	
Number of External Layers =	2	
External Layer Thickness, h_{re} =	0.2876 in	
Shim Thickness =	0.0747 in	
Number of Internal Layers =	3	
Internal Layer Thickness, h_{ri} =	0.4229 in	
Shim Thickness =	0.0747 in	
Number of Layers, n =	4	If h_{re} is ≥ h_{ri}/2 then n may be increased by 1/2 of each such layer
Shape Factor, $S = L*W / 2*h_{ri}*(L + W)$ =	8.87	Eq'n 14.7.5.1-1
Is $S^2 / n < 22$ per 14.7.6.1?	**Yes!**	If the answer is "No!", Method B must be used for design
Shear Deformation Prevented, Yes or No?	No	
Allowable Bearing Stress, σ_s =	1250.0 psi	Eq'n 14.7.6.3.2-6 & Eq'n 14.7.6.3.2-7

Maximum Bearing Stress Check

Actual Bearing Stress, R_{Total} / A =	1234.4 psi	**< Allowable…OK!**
Total Elastomeric Thickness, t =	1.844 in	**> Min. t…OK!**
Total Bearing Thickness =	**2.143 in**	
Minimum Req'd L (or W) for Stability =	6.428 in	**< L (or W)…OK!**
Shear Stiffness, K = G A / t =	12.37 kips/in	
Min. Rxn to Prevent Slip = F / μ = K Δ / 0.2 =	18.44 kips	**< Total DL Rxn…OK!**

Initial Compressive Deflection Check (This check only necessary if there is a deck joint) ** NO DECK JOINT **

	DL Only	LL Only	Total	
Compressive Stress =	778.3 psi	400.4 psi	1178.7 psi	
~~Compressive Strain, ε_c =~~	~~0.0127~~	~~0.0065~~	~~0.0201~~	~~From Eqn C14.7.5.3.6-1 or Figure C14.7.6.3.3-1~~
~~Compressive Defl., $\delta_c = \varepsilon_c t$ =~~	~~0.023 in~~	~~0.012 in~~	~~0.037 in~~	~~Eqn 14.7.5.3.6-1 & Eqn 14.7.5.3.6-2~~
~~Max. Beam Slope Prior to Slab Pour =~~	~~#REF!~~	~~Sum of Elevation Change and Camber Slope (Max all beams)~~		
~~Bevel =~~	~~-0.86%~~			
~~Actual Slope =~~	~~#REF!~~			
~~Initial Allow. Slope, $\alpha = 2\Delta_c / L$ =~~	~~0.390%~~	~~**#REF!**~~		← Ignore this check if bridge is jointless

Rotation Check

Step 1: Build profile grade into a tapered sole plate or concrete pedestal so that bearing does not have to be designed for that rotation.

Slope Due to Profile Grade, θ_G =	-0.0086 rad	+ *CCW*	From Geometry Haunch Calc.'s - Max All Beams - Instantaneous Slope at Abut.
Sole Plate MinimumThickness =	1.500 in		
Add'l Length of Sole Plate Beyond Bearing Pad =	0.500 in	(This value is applied to both sides)	
Sole Plate Maximum Thickness =	1.625 in	← Based on length of pad + add'l plate length either side, set thickness to the nearest 1/8"	

Step 2: Determine the total rotation.

Rotation for Uncertainties, θ_U =	0.0050 rad	+ *CCW*	AASHTO LRFD 14.4.2.1
Live Load Deflection, Δ_{LL} =	-1.218 in	From Beam/Girder Output	
LL Rotation, $\theta_{LL} = 4 * \Delta_{LL} / L_{span}$ =	-0.0029 rad	(or max. from girder output)	
Total Rotation, $\theta_{total} = \theta_U + \theta_{LL}$ =	0.0021 rad		

Step 3: The limits for combined compression and rotation must be satisfied to ensure that no uplift will occur.

Allowbale Stress $\sigma_S \geq 0.5*G*S*(L/h_{ri})^2*(\theta_{total}/n)$ =	239.0 psi	**< Allowable Stress…OK!**

Calculations For	Innerbelt Bridge No. 6	Job No.	49633	Sheet No.
Made by	AKS	Date	4/18/2011	
Checked by	DBT	Date	4/19/2011	
Backchecked by	AKS	Date	4/19/2011	

TITLE: HP Post Design - Rear Abutment

Shape =	HP 10x57
A_s =	16.8 in2
Height, l =	4.5 in.
r_x =	4.18 in.

DC1 Reaction =	145.60 kips	γ_{DC} =	1.25
DC2 + DW Reaction =	45.10 kips	γ_{DW} =	1.5
LL Reaction (No Impact) =	105.57 kips	γ_{LL} =	1.75

Max. Total Factored Reaction, R = 434.39 kips

Compression Check

K =	0.65	AASHTO 4.6.2.5		
F_y =	50 ksi			
E =	29000 ksi			
$\lambda = (Kl / r_x \pi)^2 * (F_y / E)$ =	0.00	<	2.25	Use AASHTO Eq'n 6.9.4.1-1

Nominal Comp. Resistance = $P_n = 0.66^{\lambda} F_y A_s$ =	839.97 kips				
ϕ_c =	0.9	AASHTO 6.5.4.2			
$P_r = \phi_c P_n$ =	755.97 kips	>	434.39 kips	OK	AASHTO 6.9.2.1

Bearing Check (AISC J7, 13th Ed.)

Actual Bearing Stress = R / A_s = 25.9 ksi

ϕ =	0.75			
Allowable Bearing Stress = ϕ 0.9 F_y =	33.8 ksi	>	25.9 ksi	OK

Rear Abutment Bearing Summary

Average Top Plate Thickness =	1.563 in.	
HP 10x57 Post Height =	4.500 in.	
Elastomer Thickness =	2.143 in.	
Bottom Load Plate Thickness =	1.500 in.	
Total Bearing Height =	9.705 in.	(enter back into Beam Seat Calcs)

Calculations For	Innerbelt Bridge No. 6	Job No.	49633	Sheet No.
Made by	AKS	Date	4/18/2011	1/10/2012
Checked by	DBT	Date	4/19/2011	1/16/2012
Backchecked by	AKS	Date	4/19/2011	1/16/2012

HNTB

TITLE: Bearing Pad Design - Forward Abutment

ELASTOMERIC BEARING PAD DESIGN USING AASHTO LRFD SECTION 14.7.6.3 - METHOD A

	Max	Min	
DC1 Reaction	145.60 kips	145.50 kips	From Beam/Girder Output
DC2 + DW Reaction	45.10 kips	41.30 kips	
LL Reaction (No Impact)	105.57 kips	96.09 kips	
Total Unfactored DL + LL Reaction, R_{Total} =	296.27 kips	282.89 kips	

Span Length, L_{span} =	138.07 ft	
Temperature Deflection, Δ_T =	0.298 in	(Temp Rise/Fall x Coeff. Therm Exp/Contr x Length x Load Factor = 60°F x 0.000006 x (L_{span}/2) x 12 in/ft x 1.0 (γ_{TG}))
Shrinkage/Creep Deflection, Δ_C =	0.249 in	(Shrinkage/Creep Coeff. x Length = (0.0005-0.0002) x (L_{span}/2) x (12 in./ft)
Min. Req'd Elast. Thick., t = 2 Δ =	1.094 in	
Pad Length (along beam), L =	12.000 in	
Pad Width (across beam), W =	20.000 in	
Durometer Hardness =	50	
For Bearing Stress Check: Shear Modulus, G =	95 psi	Based on Durometer Hardness, See Table 14.7.6.2-1
For Rotation Check: Shear Modulus, G =	130 psi	
Bearing Area, A =	240.00 in^2	
Bearing Perimeter =	64.000 in	
Number of External Layers =	2	
External Layer Thickness, h_{re} =	0.2876 in	
Shim Thickness =	0.0747 in	
Number of Internal Layers =	3	
Internal Layer Thickness, h_{ri} =	0.4229 in	
Shim Thickness =	0.0747 in	
Number of Layers, n =	4	If h_{re} is ≥ h_{ri}/2 then n may be increased by 1/2 of each such layer
Shape Factor, S = L*W / 2*h_{ri}*(L + W) =	8.87	Eq'n 14.7.5.1-1
Is S^2 / n < 22 per 14.7.6.1?	**Yes!**	If the answer is "No!", Method B must be used for design
Shear Deformation Prevented, Yes or No?	No	
Allowable Bearing Stress, σ_s =	1250.0 psi	Eq'n 14.7.6.3.2-6 & Eq'n 14.7.6.3.2-7

Maximum Bearing Stress Check

Actual Bearing Stress, R_{Total} / A =	1234.4 psi	**< Allowable…OK!**

Total Elastomeric Thickness, t =	1.844 in	**> Min. t…OK!**
Total Bearing Thickness =	**2.143 in**	
Minimum Req'd L (or W) for Stability =	6.428 in	**< L (or W)…OK!**

Shear Stiffness, K = G A / t =	12.37 kips/in	
Min. Rxn to Prevent Slip = F / μ = K Δ / 0.2 =	18.44 kips	**< Total DL Rxn…OK!**

Initial Compressive Deflection Check (This check only necessary if there is a deck joint) ** NO DECK JOINT **

	DL Only	LL Only	Total	
Compressive Stress =	778.3 psi	400.4 psi	1178.7 psi	
~~Compressive Strain, ε_c =~~	~~0.0127~~	~~0.0065~~	~~0.0201~~	~~From Eqn C14.7.5.3.6-1 or Figure C14.7.6.3.3-1~~
~~Compressive Defl., $\delta_d = \varepsilon_c$ t =~~	~~0.023 in~~	~~0.012 in~~	~~0.037 in~~	~~Eqn 14.7.5.3.6-1 & Eqn 14.7.5.3.6-2~~
~~Max. Beam Slope Prior to Slab Pour =~~	~~#REF!~~	~~Sum of Elevation Change and Camber Slope (Max all beams)~~		
~~Bevel =~~	~~2.23%~~			
~~Actual Slope =~~	~~#REF!~~			
~~Initial Allow. Slope, $\alpha = 2\Delta_c / L$ =~~	~~0.390%~~	~~#REF!~~		← Ignore this check if bridge is jointless

Rotation Check

Step 1: Build profile grade into a tapered sole plate or concrete pedestal so that bearing does not have to be designed for that rotation.

Slope Due to Profile Grade, θ_G =	0.0223 rad	+ *CW*	From Geometry Haunch Calc.'s - Max All Beams - Instantaneous Slope at Abut.
Sole Plate MinimumThickness =	1.500 in		
Add'l Length of Sole Plate Beyond Bearing Pad =	0.500 in	(This value is applied to both sides)	
Sole Plate Maximum Thickness =	1.875 in	← Based on length of pad + add'l plate length either side, set thickness to the nearest 1/8"	

Step 2: Determine the total rotation.

Rotation for Uncertainties, θ_U =	0.0050 rad	+ *CW*	AASHTO LRFD 14.4.2.1
Live Load Deflection, Δ_{LL} =	-1.218 in	From Beam/Girder Output	
LL Rotation, $\theta_{LL} = 4 * \Delta_{LL} / L_{span}$ =	-0.0029 rad	(or max. from girder output)	
Total Rotation, $\theta_{total} = \theta_U + \theta_{LL}$ =	0.0021 rad		

Step 3: The limits for combined compression and rotation must be satisfied to ensure that no uplift will occur.

Allowbale Stress $\sigma_S \geq 0.5*G*S*(L/h_{ri})^{2}*(\theta_{total}/n)$ =	239.0 psi	**< Allowable Stress…OK!**

Calculations For	Innerbelt Bridge No. 6	Job No.	49633	Sheet No.
Made by	AKS	Date	4/18/2011	
Checked by	DBT	Date	4/19/2011	
Backchecked by	AKS	Date	4/19/2011	

HNTB

TITLE: HP Post Design - Forward Abutment

Shape =	HP 10x57
A_s =	16.8 in2
Height, l =	4.5 in.
r_x =	4.18 in.

DC1 Reaction =	145.60 kips	γ_{DC} =	1.25
DC2 + DW Reaction =	45.10 kips	γ_{DW} =	1.5
LL Reaction (No Impact) =	105.57 kips	γ_{LL} =	1.75

Max. Total Factored Reaction, R = 434.39 kips

Compression Check

K =	0.65	AASHTO 4.6.2.5		
F_y =	50 ksi			
E =	29000 ksi			
$\lambda = (Kl / r_x \pi)^2 * (F_y / E)$ =	0.00	<	2.25	Use AASHTO Eq'n 6.9.4.1-1

Nominal Comp. Resistance = $P_n = 0.66^{\lambda} F_y A_s$ =	839.97 kips				
ϕ_c =	0.9	AASHTO 6.5.4.2			
$P_r = \phi_c P_n$ =	755.97 kips	>	434.39 kips	OK	AASHTO 6.9.2.1

Bearing Check (AISC J7, 13th Ed.)

Actual Bearing Stress = R / A_s = 25.9 ksi

ϕ =	0.75			
Allowable Bearing Stress = ϕ 0.9 F_y =	33.8 ksi	>	25.9 ksi	OK

Forward Abutment Bearing Summary

Average Top Plate Thickness =	1.688 in.
HP 10x57 Post Height =	4.500 in.
Elastomer Thickness =	2.143 in.
Bottom Load Plate Thickness =	1.500 in.
Total Bearing Height =	9.830 in. (enter back into Beam Seat Calcs)

1. Model geometry

This section provides model geometry information, including items such as joint coordinates, joint restraints, and element connectivity.

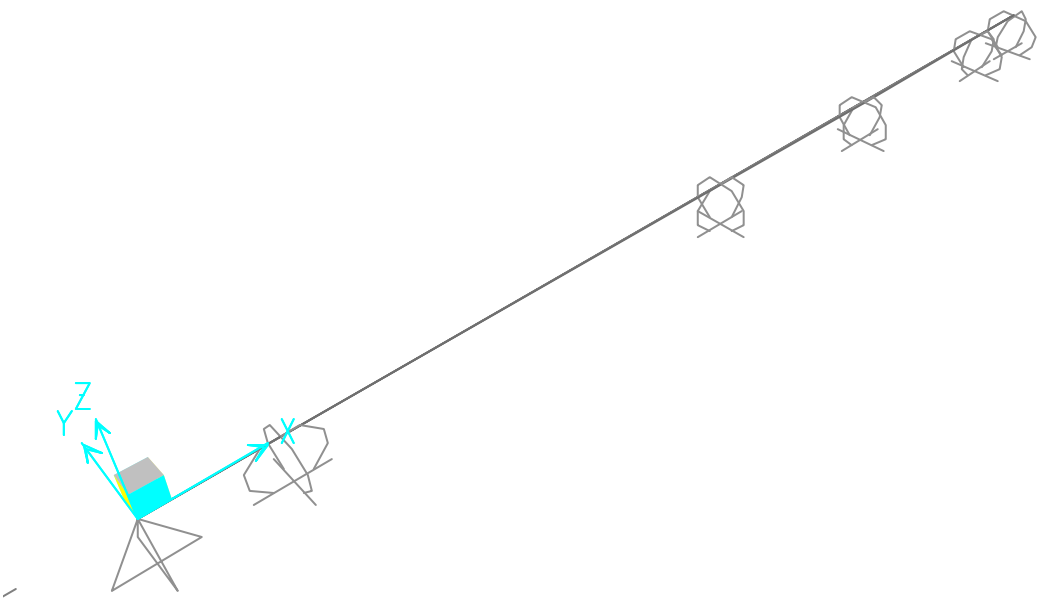


Figure 1: Finite element model

1.1. Joint coordinates

Table 1: Joint Coordinates

Table 1: Joint Coordinates

Joint	CoordSys	CoordType	GlobalX	GlobalY	GlobalZ
			ft	ft	ft
1	GLOBAL	Cartesian	0.0000	0.0000	0.0000
2	GLOBAL	Cartesian	2.6667	0.0000	0.0000
3	GLOBAL	Cartesian	14.4063	0.0000	0.0000
4	GLOBAL	Cartesian	20.0208	0.0000	0.0000
5	GLOBAL	Cartesian	25.6354	0.0000	0.0000
6	GLOBAL	Cartesian	27.5104	0.0000	0.0000

1.2. Joint restraints

Table 2: Joint Restraint Assignments

Table 2: Joint Restraint Assignments

Joint	U1	U2	U3	R1	R2	R3
1	Yes	Yes	Yes	No	No	No
2	No	No	Yes	No	No	No
3	No	No	Yes	No	No	No
4	No	No	Yes	No	No	No
5	No	No	Yes	No	No	No
6	No	No	Yes	No	No	No

1.3. Element connectivity

Table 3: Connectivity - Frame

Table 3: Connectivity - Frame

Frame	JointI	JointJ	Length ft
1	1	2	2.6667
2	2	3	11.7396
3	3	4	5.6146
4	4	5	5.6146
5	5	6	1.8750

Table 4: Frame Section Assignments

Table 4: Frame Section Assignments

Frame	AnalSect	DesignSect	MatProp
1	Footing	N.A.	Default
2	Footing	N.A.	Default
3	Footing	N.A.	Default
4	Footing	N.A.	Default
5	Footing	N.A.	Default

2. Material properties

This section provides material property information for materials used in the model.

Table 5: Material Properties 02 - Basic Mechanical Properties

Table 5: Material Properties 02 - Basic Mechanical Properties

Material	UnitWeight Kip/ft3	UnitMass Kip-s2/ft4	E1 Kip/ft2	G12 Kip/ft2	U12	A1 1/F
4000Psi	1.5000E-01	4.6621E-03	519119.50	216299.79	0.200000	5.5000E-06

Table 5: Material Properties 02 - Basic Mechanical Properties

Material	UnitWeight	UnitMass	E1	G12	U12	A1
	Kip/ft3	Kip-s2/ft4	Kip/ft2	Kip/ft2		1/F
A615Gr60	4.9000E-01	1.5230E-02	4176000.00			6.5000E-06
A992Fy50	4.9000E-01	1.5230E-02	4176000.00	1606153.85	0.300000	6.5000E-06

Table 6: Material Properties 03a - Steel Data

Table 6: Material Properties 03a - Steel Data

Material	Fy	Fu	FinalSlope
	Kip/ft2	Kip/ft2	
A992Fy50	7200.00	9360.00	-0.100000

Table 7: Material Properties 03b - Concrete Data

Table 7: Material Properties 03b - Concrete Data

Material	Fc	FinalSlope
	Kip/ft2	
4000Psi	576.00	-0.100000

Table 8: Material Properties 03e - Rebar Data

Table 8: Material Properties 03e - Rebar Data

Material	Fy	Fu	FinalSlope
	Kip/ft2	Kip/ft2	
A615Gr60	8640.00	12960.00	-0.100000

3. Section properties

This section provides section property information for objects used in the model.

3.1. Frames

Table 9: Frame Section Properties 01 - General, Part 1 of 4

Table 9: Frame Section Properties 01 - General, Part 1 of 4

SectionName	Material	Shape	t3	t2	tf	tw	t2b	tfb
			ft	ft	ft	ft	ft	ft
Footing	4000Psi	Rectangular	3.00000	6.50000				
FSEC1	A992Fy50	I/Wide Flange	1.00000	0.41667	0.03167	0.02083	0.41667	0.03167

Table 9: Frame Section Properties 01 - General, Part 2 of 4

Table 9: Frame Section Properties 01 - General, Part 2 of 4

SectionName	Area	TorsConst	I33	I22	AS2	AS3
	ft2	ft4	ft4	ft4	ft2	ft2
Footing	19.5000	41.554321	14.625000	68.656250	16.2500	16.2500
FSEC1	0.0459	0.000011	0.007615	0.000382	0.0208	0.0220

Table 9: Frame Section Properties 01 - General, Part 3 of 4

Table 9: Frame Section Properties 01 - General, Part 3 of 4

SectionName	S33	S22	Z33	Z22	R33	R22
	ft3	ft3	ft3	ft3	ft	ft
Footing	9.750000	21.125000	14.625000	31.687500	0.86603	1.87639
FSEC1	0.015230	0.001836	0.017346	0.002850	0.40730	0.09128

Table 9: Frame Section Properties 01 - General, Part 4 of 4

Table 9: Frame Section Properties 01 - General, Part 4 of 4

SectionName	AMod	A2Mod	A3Mod	JMod	I2Mod	I3Mod	MMod	WMod
Footing	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000
FSEC1	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000	1.000000

Table 10: Frame Section Properties 02 - Concrete Column, Part 1 of 2

Table 10: Frame Section Properties 02 - Concrete Column, Part 1 of 2

SectionName	RebarMatL	RebarMatC	ReinfConfig	LatReinf	Cover	NumBars3Dir	NumBars2Dir
					ft		
Footing	A615Gr60	A615Gr60	Rectangular	Ties	0.12500	3	3

Table 10: Frame Section Properties 02 - Concrete Column, Part 2 of 2

Table 10: Frame Section Properties 02 - Concrete Column, Part 2 of 2

SectionName	BarSizeL	BarSizeC	SpacingC	NumCBars2	NumCBars3
			ft		
Footing	#9	#4	0.50000	3	3

4. Load patterns

This section provides loading information as applied to the model.

4.1. Definitions

Table 11: Load Pattern Definitions

Table 11: Load Pattern Definitions

LoadPat	DesignType	SelfWtMult	AutoLoad
DEAD	DEAD	1.000000	
max vertical	OTHER	0.000000	

5. Load cases

This section provides load case information.

5.1. Definitions

Table 12: Load Case Definitions

Table 12: Load Case Definitions

Case	Type	InitialCond	ModalCase	BaseCase
DEAD	LinStatic	Zero		
MODAL	LinModal	Zero		
max vertical	LinStatic	Zero		

5.2. Static case load assignments

Table 13: Case - Static 1 - Load Assignments

Table 13: Case - Static 1 - Load Assignments

Case	LoadType	LoadName	LoadSF
DEAD	Load pattern	DEAD	1.000000
max vertical	Load pattern	max vertical	1.000000

5.3. Response spectrum case load assignments

Table 14: Function - Response Spectrum - User

Table 14: Function - Response Spectrum - User

Name	Period	Accel	FuncDamp
	Sec		
UNIFRS	0.000000	1.000000	0.050000
UNIFRS	1.000000	1.000000	

6. Load combinations

This section provides load combination information.

Table 15: Combination Definitions

Table 15: Combination Definitions

ComboName	ComboType	CaseName	ScaleFactor
COMB1	Linear Add	DEAD	1.000000
COMB1		max vertical	1.000000

7. Structure results

This section provides structure results, including items such as structural periods and base reactions.

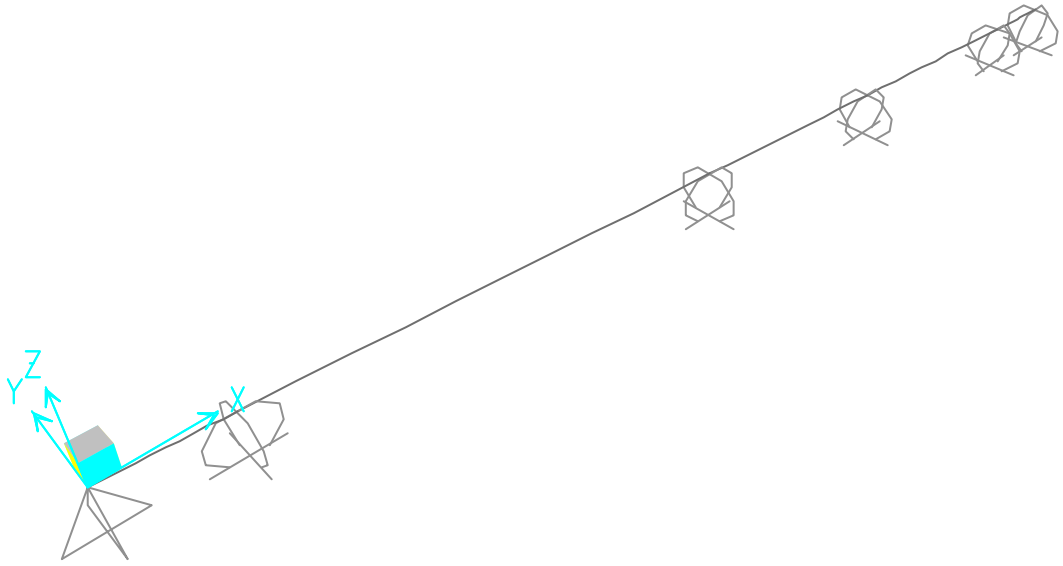


Figure 2: Deformed shape

7.1. Mass summary

Table 16: Assembled Joint Masses

Table 16: Assembled Joint Masses

Joint	U1	U2	U3	R1	R2	R3
	Kip-s2/ft	Kip-s2/ft	Kip-s2/ft	Kip-ft-s2	Kip-ft-s2	Kip-ft-s2
1	0.12	0.12	0.12	0.0000	0.0000	0.0000
2	0.65	0.65	0.65	0.0000	0.0000	0.0000
3	0.79	0.79	0.79	0.0000	0.0000	0.0000
4	0.51	0.51	0.51	0.0000	0.0000	0.0000
5	0.34	0.34	0.34	0.0000	0.0000	0.0000
6	8.523E-02	8.523E-02	8.523E-02	0.0000	0.0000	0.0000

7.2. Modal results

Table 17: Modal Participating Mass Ratios

Table 17: Modal Participating Mass Ratios

OutputCase	StepNum	Period	UX	UY	UZ	SumUX	SumUY	SumUZ
		Sec						
MODAL	1.000000	354790.127	0.00000	0.75212	0.00000	0.00000	0.75212	0.00000
MODAL	2.000000	300378.9633	0.00118	0.00122	0.00000	0.00118	0.75335	0.00000

Table 17: Modal Participating Mass Ratios

OutputCase	StepNum	Period	UX	UY	UZ	SumUX	SumUY	SumUZ
		Sec						
MODAL	3.000000	-288062.223	0.30880	0.09476	0.00000	0.30998	0.84811	0.00000
MODAL	4.000000	210280.5608	0.43478	0.01014	0.00000	0.74476	0.85825	0.00000
MODAL	5.000000	173875.8226	0.08224	0.02978	0.00000	0.82700	0.88803	0.00000
MODAL	6.000000	159880.0758	0.07430	0.01251	0.00000	0.90130	0.90054	0.00000
MODAL	7.000000	145710.4730	0.01731	0.00180	0.00000	0.91861	0.90234	0.00000
MODAL	8.000000	66637.8158	0.05829	0.02459	0.00000	0.97690	0.92693	0.00000
MODAL	9.000000	24121.19576	0.02298	0.07187	0.00000	0.99989	0.99879	0.00000
MODAL	10.000000	6726.314510	0.00011	0.00121	0.00000	1.00000	1.00000	0.00000

7.3. Base reactions

Table 18: Base Reactions

Table 18: Base Reactions

OutputCase	GlobalFX	GlobalFY	GlobalFZ	GlobalMX	GlobalMY	GlobalMZ
	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft
DEAD	0.000	0.000	80.468	0.0000	-1106.8537	0.0000
max vertical	0.000	0.000	1688.314	0.0000	-23223.1145	0.0000

8. Joint results

This section provides joint results, including items such as displacements and reactions.

Table 19: Joint Displacements

Table 19: Joint Displacements

Joint	OutputCase	U1	U2	U3	R1	R2	R3
		ft	ft	ft	Radians	Radians	Radians
1	DEAD	0.000000	0.000000	0.000000	0.000000	1.570E-06	0.000000
1	max vertical	0.000000	0.000000	0.000000	0.000000	0.000033	0.000000
2	DEAD	0.000000	0.000000	0.000000	0.000000	5.577E-06	0.000000
2	max vertical	0.000000	0.000000	0.000000	0.000000	0.000117	0.000000
3	DEAD	0.000000	0.000000	0.000000	0.000000	-5.480E-06	0.000000
3	max vertical	0.000000	0.000000	0.000000	0.000000	-0.000115	0.000000
4	DEAD	0.000000	0.000000	0.000000	0.000000	7.925E-07	0.000000
4	max vertical	0.000000	0.000000	0.000000	0.000000	0.000017	0.000000
5	DEAD	0.000000	0.000000	0.000000	0.000000	-1.009E-06	0.000000
5	max vertical	0.000000	0.000000	0.000000	0.000000	-0.000021	0.000000
6	DEAD	0.000000	0.000000	0.000000	0.000000	-6.327E-07	0.000000
6	max vertical	0.000000	0.000000	0.000000	0.000000	-0.000013	0.000000

Table 20: Joint Reactions

Table 20: Joint Reactions

Joint	OutputCase	F1	F2	F3	M1	M2	M3
		Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft
1	DEAD	0.000	0.000	-5.956	0.0000	0.0000	0.0000
1	max vertical	0.000	0.000	-124.973	0.0000	0.0000	0.0000
2	DEAD	0.000	0.000	30.899	0.0000	0.0000	0.0000
2	max vertical	0.000	0.000	648.288	0.0000	0.0000	0.0000
3	DEAD	0.000	0.000	29.125	0.0000	0.0000	0.0000
3	max vertical	0.000	0.000	611.071	0.0000	0.0000	0.0000
4	DEAD	0.000	0.000	12.877	0.0000	0.0000	0.0000
4	max vertical	0.000	0.000	270.184	0.0000	0.0000	0.0000
5	DEAD	0.000	0.000	13.323	0.0000	0.0000	0.0000
5	max vertical	0.000	0.000	279.524	0.0000	0.0000	0.0000
6	DEAD	0.000	0.000	0.201	0.0000	0.0000	0.0000
6	max vertical	0.000	0.000	4.219	0.0000	0.0000	0.0000

9. Frame results

This section provides frame force results.

Table 21: Element Forces - Frames, Part 1 of 2

Table 21: Element Forces - Frames, Part 1 of 2

Frame	Station	OutputCase	P	V2	V3
	ft		Kip	Kip	Kip
1	0.0000	DEAD	0.000	5.956	0.000
1	1.3333	DEAD	0.000	9.856	0.000
1	2.6667	DEAD	0.000	13.756	0.000
1	0.0000	max vertical	0.000	124.973	0.000
1	1.3333	max vertical	0.000	206.799	0.000
1	2.6667	max vertical	0.000	288.626	0.000
2	0.0000	DEAD	0.000	-17.142	0.000
2	1.9566	DEAD	0.000	-11.419	0.000
2	3.9132	DEAD	0.000	-5.696	0.000
2	5.8698	DEAD	0.000	0.027	0.000
2	7.8264	DEAD	0.000	5.750	0.000
2	9.7830	DEAD	0.000	11.473	0.000
2	11.7396	DEAD	0.000	17.196	0.000
2	0.0000	max vertical	0.000	-359.662	0.000
2	1.9566	max vertical	0.000	-239.586	0.000
2	3.9132	max vertical	0.000	-119.510	0.000
2	5.8698	max vertical	0.000	0.567	0.000
2	7.8264	max vertical	0.000	120.643	0.000
2	9.7830	max vertical	0.000	240.720	0.000
2	11.7396	max vertical	0.000	360.796	0.000
3	0.0000	DEAD	0.000	-11.929	0.000
3	1.8715	DEAD	0.000	-6.454	0.000
3	3.7431	DEAD	0.000	-0.980	0.000
3	5.6146	DEAD	0.000	4.494	0.000
3	0.0000	max vertical	0.000	-250.275	0.000
3	1.8715	max vertical	0.000	-135.420	0.000

Table 21: Element Forces - Frames, Part 1 of 2

Frame	Station	OutputCase	P	V2	V3
	ft		Kip	Kip	Kip
3	3.7431	max vertical	0.000	-20.564	0.000
3	5.6146	max vertical	0.000	94.292	0.000
4	0.0000	DEAD	0.000	-8.383	0.000
4	1.8715	DEAD	0.000	-2.909	0.000
4	3.7431	DEAD	0.000	2.565	0.000
4	5.6146	DEAD	0.000	8.039	0.000
4	0.0000	max vertical	0.000	-175.892	0.000
4	1.8715	max vertical	0.000	-61.037	0.000
4	3.7431	max vertical	0.000	53.819	0.000
4	5.6146	max vertical	0.000	168.675	0.000
5	0.0000	DEAD	0.000	-5.283	0.000
5	1.8750	DEAD	0.000	0.201	0.000
5	0.0000	max vertical	0.000	-110.849	0.000
5	1.8750	max vertical	0.000	4.219	0.000

Table 21: Element Forces - Frames, Part 2 of 2

Table 21: Element Forces - Frames, Part 2 of 2

Frame	Station	OutputCase	T	M2	M3
	ft		Kip-ft	Kip-ft	Kip-ft
1	0.0000	DEAD	0.0000	0.0000	-1.184E-15
1	1.3333	DEAD	0.0000	0.0000	-10.5419
1	2.6667	DEAD	0.0000	0.0000	-26.2838
1	0.0000	max vertical	0.0000	0.0000	-1.895E-14
1	1.3333	max vertical	0.0000	0.0000	-221.1813
1	2.6667	max vertical	0.0000	0.0000	-551.4648
2	0.0000	DEAD	0.0000	0.0000	-26.2838
2	1.9566	DEAD	0.0000	0.0000	1.6576
2	3.9132	DEAD	0.0000	0.0000	18.4013
2	5.8698	DEAD	0.0000	0.0000	23.9473
2	7.8264	DEAD	0.0000	0.0000	18.2956
2	9.7830	DEAD	0.0000	0.0000	1.4462
2	11.7396	DEAD	0.0000	0.0000	-26.6009
2	0.0000	max vertical	0.0000	0.0000	-551.4648
2	1.9566	max vertical	0.0000	0.0000	34.7790
2	3.9132	max vertical	0.0000	0.0000	386.0816
2	5.8698	max vertical	0.0000	0.0000	502.4431
2	7.8264	max vertical	0.0000	0.0000	383.8636
2	9.7830	max vertical	0.0000	0.0000	30.3429
2	11.7396	max vertical	0.0000	0.0000	-558.1188
3	0.0000	DEAD	0.0000	0.0000	-26.6009
3	1.8715	DEAD	0.0000	0.0000	-9.3989
3	3.7431	DEAD	0.0000	0.0000	-2.4420
3	5.6146	DEAD	0.0000	0.0000	-5.7303
3	0.0000	max vertical	0.0000	0.0000	-558.1188
3	1.8715	max vertical	0.0000	0.0000	-197.1996
3	3.7431	max vertical	0.0000	0.0000	-51.2360
3	5.6146	max vertical	0.0000	0.0000	-120.2279
4	0.0000	DEAD	0.0000	0.0000	-5.7303
4	1.8715	DEAD	0.0000	0.0000	4.8368
4	3.7431	DEAD	0.0000	0.0000	5.1587
4	5.6146	DEAD	0.0000	0.0000	-4.7645
4	0.0000	max vertical	0.0000	0.0000	-120.2279

Table 21: Element Forces - Frames, Part 2 of 2

Frame	Station	OutputCase	T	M2	M3
	ft		Kip-ft	Kip-ft	Kip-ft
4	1.8715	max vertical	0.0000	0.0000	101.4817
4	3.7431	max vertical	0.0000	0.0000	108.2357
4	5.6146	max vertical	0.0000	0.0000	-99.9658
5	0.0000	DEAD	0.0000	0.0000	-4.7645
5	1.8750	DEAD	0.0000	0.0000	1.776E-15
5	0.0000	max vertical	0.0000	0.0000	-99.9658
5	1.8750	max vertical	0.0000	0.0000	0.0000

10. Material take-off

This section provides a material take-off.

Table 22: Material List 2 - By Section Property

Table 22: Material List 2 - By Section Property

Section	ObjectType	NumPieces	TotalLength	TotalWeight
			ft	Kip
Footing	Frame	5	27.5104	80.468

11. Design preferences

This section provides the design preferences for each type of design, which typically include material reduction factors, framing type, stress ratio limit, deflection limits, and other code specific items.

11.1. Steel design

Table 23: Preferences - Steel Design - AISC-LRFD93, Part 1 of 2

Table 23: Preferences - Steel Design - AISC-LRFD93, Part 1 of 2

FrameType	PatLLF	SRatioLimit	PhiB	PhiC	PhiT	PhiV	PhiCA
Moment Frame	0.750000	0.950000	0.900000	0.850000	0.900000	0.900000	0.900000

Table 23: Preferences - Steel Design - AISC-LRFD93, Part 2 of 2

Table 23: Preferences - Steel Design - AISC-LRFD93, Part 2 of 2

DLRat	SDLAndLLRat	LLRat	TotalRat	NetRat
120.000000	120.000000	360.000000	240.000000	240.000000

11.2. Concrete design

Table 24: Preferences - Concrete Design - ACI 318-05/IBC2003, Part 1 of 2

Table 24: Preferences - Concrete Design - ACI 318-05/IBC2003, Part 1 of 2

MinEccen	PatLLF	UFLimit	SeisCat	PhiT
Yes	0.750000	0.950000	D	0.900000

Table 24: Preferences - Concrete Design - ACI 318-05/IBC2003, Part 2 of 2

Table 24: Preferences - Concrete Design - ACI 318-05/IBC2003, Part 2 of 2

PhiCTied	PhiCSpiral	PhiV	PhiVSeismic	PhiVJoint
0.650000	0.700000	0.750000	0.600000	0.850000

11.3. Aluminum design

Table 25: Preferences - Aluminum Design - AA-ASD 2000

Table 25: Preferences - Aluminum Design - AA-ASD 2000

FrameType	SRatioLimit	LatFact	UseLatFact
Moment Frame	1.000000	1.333333	No

11.4. Cold formed design

Table 26: Preferences - Cold Formed Design - AISI-ASD96

Table 26: Preferences - Cold Formed Design - AISI-ASD96

FrameType	SRatioLimit	OmegaBS	OmegaBUS	OmegaBLTB	OmegaVS	OmegaVNS	OmegaT	OmegaC
Braced Frame	1.000000	1.670000	1.670000	1.670000	1.670000	1.500000	1.670000	1.800000

Table: Element Forces - Frames, Part 1 of 2

Table: Element Forces - Frames, Part 1 of 2

Frame	Station	OutputCase	CaseType	P	V2	V3	T	M2
	ft			Kip	Kip	Kip	Kip-ft	Kip-ft
1	0.0000	COMB1	Combination	0.000	130.929	0.000	0.0000	0.0000
1	1.3333	COMB1	Combination	0.000	216.656	0.000	0.0000	0.0000
1	2.6667	COMB1	Combination	0.000	302.382	0.000	0.0000	0.0000
2	0.0000	COMB1	Combination	0.000	-376.804	0.000	0.0000	0.0000
2	1.9566	COMB1	Combination	0.000	-251.005	0.000	0.0000	0.0000
2	3.9132	COMB1	Combination	0.000	-125.206	0.000	0.0000	0.0000
2	5.8698	COMB1	Combination	0.000	0.594	0.000	0.0000	0.0000
2	7.8264	COMB1	Combination	0.000	126.393	0.000	0.0000	0.0000
2	9.7830	COMB1	Combination	0.000	252.193	0.000	0.0000	0.0000
2	11.7396	COMB1	Combination	0.000	377.992	0.000	0.0000	0.0000
3	0.0000	COMB1	Combination	0.000	-262.204	0.000	0.0000	0.0000
3	1.8715	COMB1	Combination	0.000	-141.874	0.000	0.0000	0.0000
3	3.7431	COMB1	Combination	0.000	-21.544	0.000	0.0000	0.0000
3	5.6146	COMB1	Combination	0.000	98.786	0.000	0.0000	0.0000
4	0.0000	COMB1	Combination	0.000	-184.276	0.000	0.0000	0.0000
4	1.8715	COMB1	Combination	0.000	-63.946	0.000	0.0000	0.0000
4	3.7431	COMB1	Combination	0.000	56.384	0.000	0.0000	0.0000
4	5.6146	COMB1	Combination	0.000	176.714	0.000	0.0000	0.0000
5	0.0000	COMB1	Combination	0.000	-116.133	0.000	0.0000	0.0000
5	1.8750	COMB1	Combination	0.000	4.420	0.000	0.0000	0.0000

Table: Element Forces - Frames, Part 2 of 2

Table: Element Forces - Frames, Part 2 of 2

Frame	Station	OutputCase	M3	FrameElem	ElemStation
	ft		Kip-ft		ft
1	0.0000	COMB1	-2.013E-14	1-1	0.0000
1	1.3333	COMB1	-231.7232	1-1	1.3333
1	2.6667	COMB1	-577.7485	1-1	2.6667
2	0.0000	COMB1	-577.7485	2-1	0.0000
2	1.9566	COMB1	36.4366	2-1	1.9566
2	3.9132	COMB1	404.4829	2-1	3.9132
2	5.8698	COMB1	526.3904	2-1	5.8698
2	7.8264	COMB1	402.1592	2-1	7.8264
2	9.7830	COMB1	31.7891	2-1	9.7830
2	11.7396	COMB1	-584.7197	2-1	11.7396
3	0.0000	COMB1	-584.7197	3-1	0.0000
3	1.8715	COMB1	-206.5985	3-1	1.8715
3	3.7431	COMB1	-53.6780	3-1	3.7431
3	5.6146	COMB1	-125.9581	3-1	5.6146
4	0.0000	COMB1	-125.9581	4-1	0.0000
4	1.8715	COMB1	106.3185	4-1	1.8715
4	3.7431	COMB1	113.3944	4-1	3.7431
4	5.6146	COMB1	-104.7304	4-1	5.6146
5	0.0000	COMB1	-104.7304	5-1	0.0000
5	1.8750	COMB1	1.776E-15	5-1	1.8750

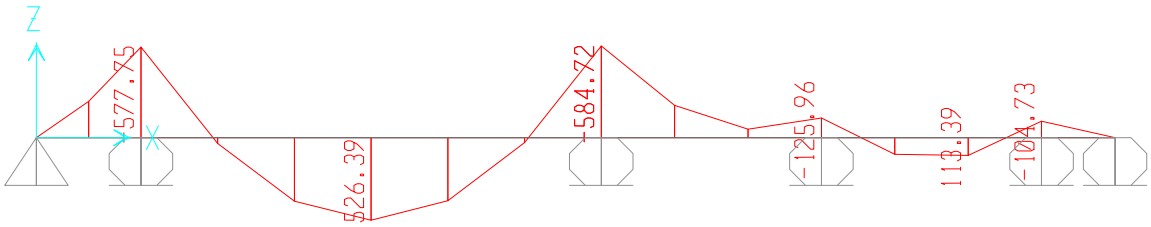

Z
X
577.75
-26.39
-584.72
-125.96
113.39
-104.73

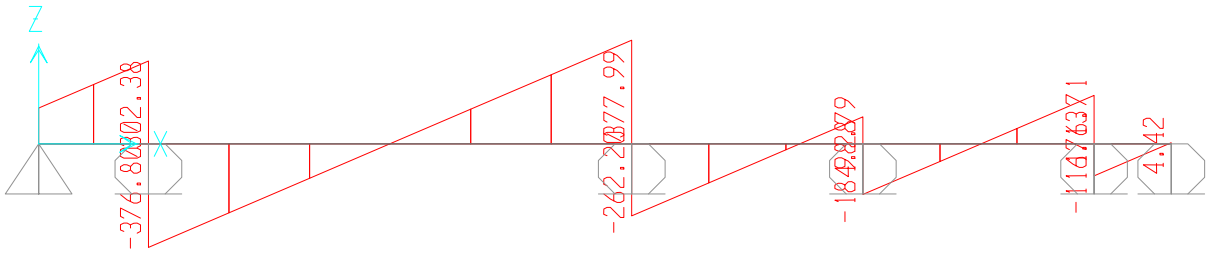
TABLE: Element Forces - Frames

Frame	Station	OutputCase	CaseType	P	V2	V3	T	M2	M3	FrameElem	ElemStation
Text	in	Text	Text	Kip	Kip	Kip	Kip-in	Kip-in	Kip-in	Text	in
1	0	COMB1	Combination	0	130.929036	0	0	0	-2.42E-13	1-Jan	0
1	16	COMB1	Combination	0	216.6557027	0	0	0	-2780.67791	1-Jan	16
1	32	COMB1	Combination	0	302.3823694	0	0	0	-6932.982487	1-Jan	32
2	0	COMB1	Combination	0	-376.8044369	0	0	0	-6932.982487	1-Feb	0
2	23.47916667	COMB1	Combination	0	-251.0050185	0	0	0	437.2389314	1-Feb	23.47916667
2	46.95833333	COMB1	Combination	0	-125.2056001	0	0	0	4853.794838	1-Feb	46.95833333
2	70.4375	COMB1	Combination	0	0.593818344	0	0	0	6316.685234	1-Feb	70.4375
2	93.91666667	COMB1	Combination	0	126.3932367	0	0	0	4825.910119	1-Feb	93.91666667
2	117.3958333	COMB1	Combination	0	252.1926551	0	0	0	381.469492	1-Feb	117.3958333
2	140.875	COMB1	Combination	0	377.9920736	0	0	0	-7016.636646	1-Feb	140.875
3	0	COMB1	Combination	0	-262.2037452	0	0	0	-7016.636646	1-Mar	0
3	22.45833333	COMB1	Combination	0	-141.8738668	0	0	0	-2479.181794	1-Mar	22.45833333
3	44.91666667	COMB1	Combination	0	-21.5439883	0	0	0	-644.1354636	1-Mar	44.91666667
3	67.375	COMB1	Combination	0	98.78589017	0	0	0	-1511.497653	1-Mar	67.375
4	0	COMB1	Combination	0	-184.2756423	0	0	0	-1511.497653	1-Apr	0
4	22.45833333	COMB1	Combination	0	-63.94576386	0	0	0	1275.821887	1-Apr	22.45833333
4	44.91666667	COMB1	Combination	0	56.38411461	0	0	0	1360.732907	1-Apr	44.91666667
4	67.375	COMB1	Combination	0	176.7139931	0	0	0	-1256.764594	1-Apr	67.375
5	0	COMB1	Combination	0	-116.1327667	0	0	0	-1256.764594	1-May	0
5	22.5	COMB1	Combination	0	4.420358312	0	0	0	2.13E-14	1-May	22.5

TABLE: Element Joint Forces - Frames

Frame	Joint	OutputCase	CaseType	F1	F2	F3	M1	M2	M3	FrameElem
Text	Text	Text	Text	Kip	Kip	Kip	Kip-in	Kip-in	Kip-in	Text
1	1	COMB1	Combination	0	0	-130.929036	0	-2.45E-13	0	1-Jan
1	2	COMB1	Combination	0	0	302.3823694	0	6932.982487	0	1-Jan
2	2	COMB1	Combination	0	0	376.8044369	0	-6932.982487	0	1-Feb
2	3	COMB1	Combination	0	0	377.9920736	0	7016.636646	0	1-Feb
3	3	COMB1	Combination	0	0	262.2037452	0	-7016.636646	0	1-Mar
3	4	COMB1	Combination	0	0	98.78589017	0	1511.497653	0	1-Mar
4	4	COMB1	Combination	0	0	184.2756423	0	-1511.497653	0	1-Apr
4	5	COMB1	Combination	0	0	176.7139931	0	1256.764594	0	1-Apr
5	5	COMB1	Combination	0	0	116.1327667	0	-1256.764594	0	1-May
5	6	COMB1	Combination	0	0	4.420358312	0	1.10E-13	0	1-May

TABLE: Program Control															
ProgramName	Version	ProgLevel	LicenseNum	LicenseOS	LicenseSC	LicenseBR	LicenseHT	CurrUnits	SteelCode	ConcCode	AlumCode	ColdCode	BridgeCode	RegenHinge	BSchedGUID
Text	Text	Text	Text	Yes/No	Yes/No	Yes/No	Yes/No	Text	Text	Text	Text	Text	Text	Yes/No	Text
SAP2000	14.2.4	Advanced	21A5C	No	Yes	Yes	No	Kip, in, F	AISC-LRFD93	ACI 318-05/IBC2003	AA-ASD 2000	AISI-ASD96	AASHTO LRFD 2007	Yes	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4.42

HAUNCH AND BEARINGS

Calculations For	Innerbelt Bridge No. 6	Job No.	49633	Sheet No.
Made by	AKS	Date	4/18/2011	
Checked by	DBT	Date	4/19/2011	
Backchecked by	AKS	Date	4/19/2011	

HNTB

Title: Beam Seat / Haunch / Bearing Process

Determine deck elevations at ends and midspan.

Gather deflections and cambers from Conspan.

Using haunch spreadsheet enter deck elevations and properties, initial cambers, mid-span deflections, span lengths, and beam properties.

- Enter a placeholder / estimate for the bearing height.
- Revise haunch at ends until minimum allowable check is satisfied.
- Copy tab, set mid-span deflections to zero, and insert the residual camber in place of the initial camber in order to get slopes.

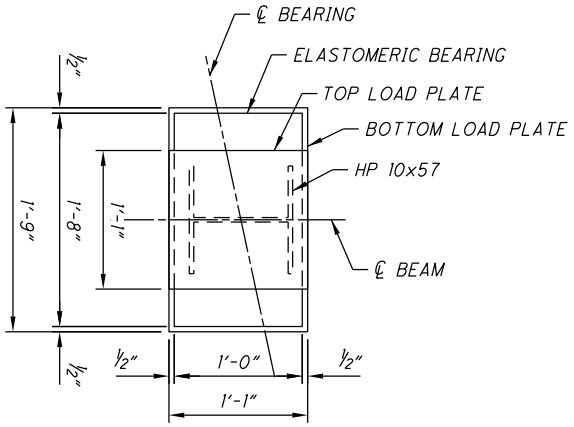
Gather reactions from Conspan; actual slopes.

Using bearing spreadsheet, enter reactions, maximum and minimum (no Live Load on minimum).

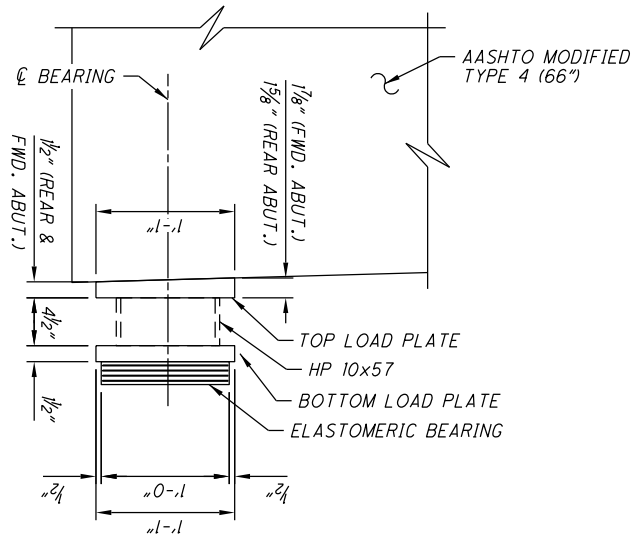
Go back to haunch sheet and input correct bearing lengths.

BEARING DESIGN SUMMARY - BRIDGE 6

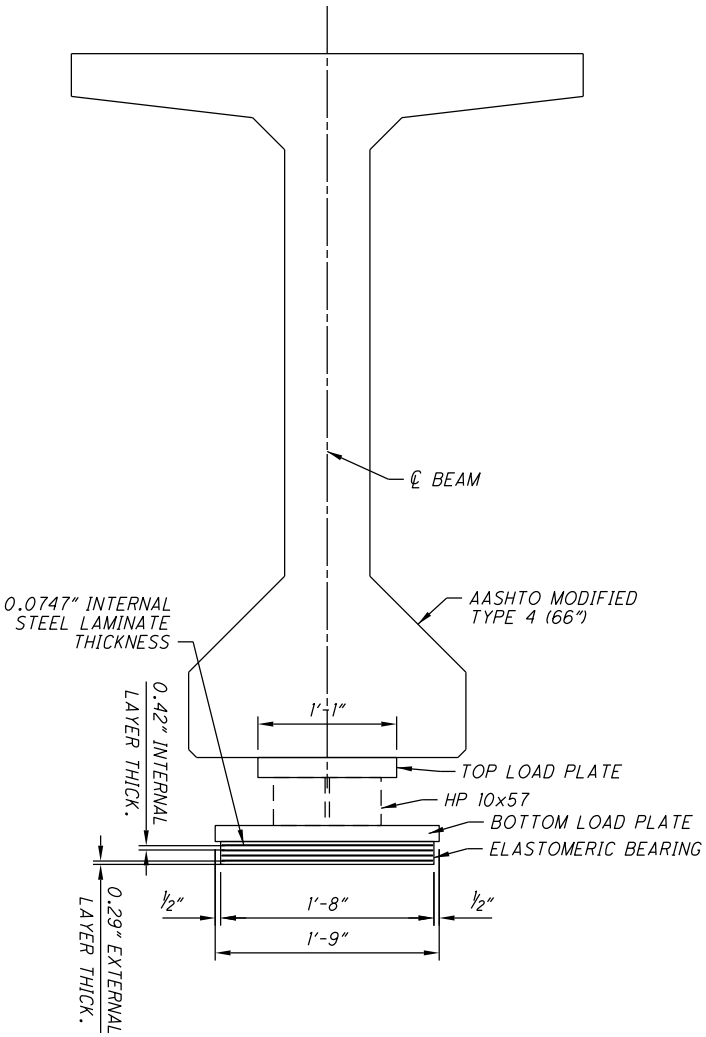
BY:	AKS	4/18/11
CHECK:	DBT	4/19/11
BKCHECK:	AKS	4/19/11
VERIFIED:		



BEARING PLAN



BEARING ELEVATION



SECTION A-A

Calculations For	Innerbelt Bridge No. 6	Job No.	49633	Sheet No.
Made by	AKS	Date	4/18/2011	1/10/2012
Checked by	DBT	Date	4/19/2011	1/16/2012
Backchecked by	AKS	Date	4/19/2011	1/16/2012

HNTB

Title: Conspan Output for Haunch Calculations

	(To Calculate Beam Seat Elevation) *									
	Prestress	Self Wt.	**Init. Camber**	Deck+Haunch	DL-Prec. (DC)	Diaphragm	DL-Comp. (DC)	DL-Comp. (DW)	**Mid-Span Defl.**	Total**
Span 1										
Beam 1	11.893	-5.643	6.250	-1.801	-0.804	-0.018	-0.170	-0.629	-3.422	2.828
Beam 2	11.893	-5.643	6.250	-1.971	-0.620	-0.036	-0.181	-0.670	-3.478	2.772
								Place Maximum Value in Haunch Spreadsheet -->	**-3.478**	

* Use Erection Deflections @ 0.5L

**For Initial Camber when Rotation = 0

CONSPAN values are opposite in the Haunch & Beam Seats Spreadsheet: (+) Downward, (-) Upward

Calculations For	Innerbelt Bridge No. 6	Job No.	49633	Sheet No.
Made by	AKS	Date	4/18/2011	1/10/2012
Checked by	DBT	Date	4/19/2011	1/16/2012
Backchecked by	AKS	Date	4/19/2011	1/16/2012

HNTB

Title: Haunch Thickness & Beam Seat Calculation - Beams 1 thru 5

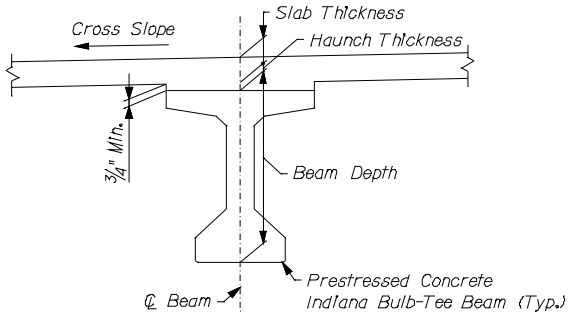
Units ("in" or "mm")	in	
Slab Thickness	8.50 in	*CAMBER: (-)=Upward, (+)=Downward
Beam Depth	66.00 in	
CL Bm to Top Flange Edge	24.00 in	(See Sketch Below for Clarity of Values)

Mid-Span Non-Comp. & Composite DL Deflections

Span 1
3.478 in

These values are added into Mid-Span Elevations

Location	Top Deck Elevation (ft)	Haunch at Ends (in)	Initial Camber* (in)	Top Beam Elevation (ft)	Net Haunch (in)	Min. Allow Haunch (in)	Bearing Height (in)	Bm Seat Elevation (ft)	Span Length (ft)	Beam Slope (%)	Camber Slope (%)
Span 1 - WB Beam 1											
Rear Abut	699.275	4.5		698.192			9.705	**691.883**		-0.03%	
Midspan	698.686		-6.25	697.653	3.89	1.49			137.375	-1.54%	-1.52%
Forward Abut	697.198	5		696.073		**OK!**	9.83	**689.754**		-3.06%	
Span 1 - WB Beam 2											
Rear Abut	699.153	4.5		698.070			9.705	**691.761**		0.13%	
Midspan	698.601		-6.25	697.638	3.06	1.37			137.375	-1.39%	-1.52%
Forward Abut	697.289	5		696.164		**OK!**	9.83	**689.845**		-2.90%	
Span 1 - WB Beam 3											
Rear Abut	698.790	4.5		697.707			9.705	**691.398**		0.25%	
Midspan	698.290		-6.25	697.356	2.71	1.33			137.375	-1.27%	-1.52%
Forward Abut	697.088	5		695.963		**OK!**	9.83	**689.644**		-2.79%	
Span 1 - WB Beam 4											
Rear Abut	698.580	4.5		697.497			9.705	**691.188**		0.26%	
Midspan	698.049		-6.25	697.153	2.25	1.33			138.070	-1.25%	-1.51%
Forward Abut	696.892	5		695.767		**OK!**	9.83	**689.448**		-2.76%	
Span 1 - WB Beam 5											
Rear Abut	698.210	4.5		697.127			9.705	**690.818**		0.38%	
Midspan	697.743		-6.25	696.869	1.98	1.32			138.070	-1.13%	-1.51%
Forward Abut	696.695	5		695.570		**OK!**	9.83	**689.251**		-2.64%	



Calculations For	Innerbelt Bridge No. 6	Job No.	49633	Sheet No.
Made by	AKS	Date	4/18/2011	1/10/2012
Checked by	DBT	Date	4/19/2011	1/16/2012
Backchecked by	AKS	Date	4/19/2011	1/16/2012

HNTB

Title: Haunch Thickness & Beam Seat Calculation - Beams 6 thru 10

Units ("in" or "mm")	in	
Slab Thickness	8.50 in	*CAMBER: (-)=Upward, (+)=Downward
Beam Depth	66.00 in	
CL Bm to Top Flange Edge	24.00 in	(See Sketch Below for Clarity of Values)

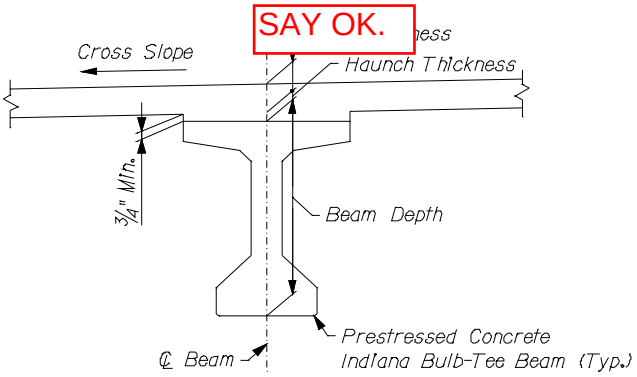
Mid-Span Non-Comp. & Composite DL Deflections

Span 1
3.478 in

These values are added into Mid-Span Elevations

Location	Top Deck Elevation (ft)	Haunch at Ends (in)	Initial Camber* (in)	Top Beam Elevation (ft)	Net Haunch (in)	Min. Allow Haunch (in)	Bearing Height (in)	Bm Seat Elevation (ft)	Span Length (ft)	Beam Slope (%)	Camber Slope (%)
Span 1 - WB Beam 6											
Rear Abut	697.840	4.5		696.757			9.705	**690.448**		0.51%	
Midspan	697.443		-6.25	696.589	1.75	1.27			138.070	-1.00%	-1.51%
Forward Abut	696.504	5		695.379		**OK!**	9.83	**689.060**		-2.51%	
Span 1 - WB Beam 7											
Rear Abut	697.470	4.5		696.387			9.705	**690.078**		0.64%	
Midspan	697.154		-6.25	696.307	1.67	1.27			138.070	-0.87%	-1.51%
Forward Abut	696.310	5		695.185		**OK!**	9.83	**688.866**		-2.38%	
Span 1 - WB Beam 8											
Rear Abut	697.100	4.5		696.017			9.705	**689.708**		0.78%	
Midspan	696.864		-6.25	696.037	1.43	1.27			138.070	-0.73%	-1.51%
Forward Abut	696.140	5		695.015		**OK!**	9.83	**688.696**		-2.23%	
Span 1 - WB Beam 9											
Rear Abut	696.730	4.5		695.647			9.705	**689.338**		0.93%	
Midspan	696.580		-6.25	695.765	1.28	1.23			138.070	-0.58%	-1.51%
Forward Abut	695.967	5		694.842		**OK!**	9.83	**688.523**		-2.09%	
Span 1 - WB Beam 10											
Rear Abut	696.360	4.5		695.277			9.705	**688.968**		1.07%	
Midspan	696.305		-6.25	695.498	1.19	1.24			138.070	-0.43%	-1.51%
Forward Abut	695.802	5		694.677	**NEED MORE HAUNCH!**		9.83	**688.358**		-1.94%	

SAY OK.



Calculations For	Innerbelt Bridge No. 6	Job No.	49633	Sheet No.
Made by	AKS	Date	4/18/2011	1/10/2012
Checked by	DBT	Date	4/19/2011	1/16/2012
Backchecked by	AKS	Date	4/19/2011	1/16/2012

HNTB

Title: Rotation Calculation, no DL Deflection - Beams 1 thru 5

Units ("in" or "mm") in
Slab Thickness 8.50 in
Beam Depth 66.00 in
CL Bm to Top Flange Edge 24.00 in

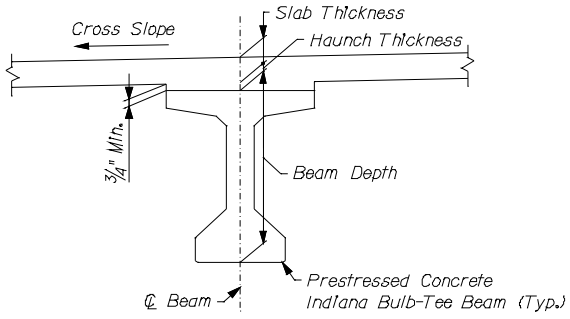
*CAMBER: (-)=Upward, (+)=Downward
(See Sketch Below for Clarity of Values)

Mid-Span Non-Comp. & Composite DL Deflections

Span 1
0.000 in

These values are added into Mid-Span Elevations

Location	Top Deck Elevation (ft)	Haunch at Ends (in)	Initial Camber* (in)	Top Beam Elevation (ft)	Net Haunch (in)	Min. Allow Haunch (in)	Bearing Height (in)	Bm Seat Elevation (ft)	Span Length (ft)	Beam Slope (%)	Camber Slope (%)
Span 1 - WB Beam 1											
Rear Abut	699.275	4.5		698.192			9.705	**691.883**		-0.86%	
Midspan	698.396		-2.828	697.368	3.84	1.49			137.375	-1.54%	-0.69%
Forward Abut	697.198	5		696.073		**OK!**	9.83	**689.754**		-2.23%	
Span 1 - WB Beam 2											
Rear Abut	699.153	4.5		698.070			9.705	**691.761**		-0.71%	
Midspan	698.311		-2.772	697.348	3.06	1.37			137.375	-1.39%	-0.67%
Forward Abut	697.289	5		696.164		**OK!**	9.83	**689.845**		-2.06%	
Span 1 - WB Beam 3											
Rear Abut	698.790	4.5		697.707			9.705	**691.398**		-0.60%	
Midspan	698.000		-2.772	697.066	2.71	1.33			137.375	-1.27%	-0.67%
Forward Abut	697.088	5		695.963		**OK!**	9.83	**689.644**		-1.94%	
Span 1 - WB Beam 4											
Rear Abut	698.580	4.5		697.497			9.705	**691.188**		-0.58%	
Midspan	697.759		-2.772	696.863	2.25	1.33			138.070	-1.25%	-0.67%
Forward Abut	696.892	5		695.767		**OK!**	9.83	**689.448**		-1.92%	
Span 1 - WB Beam 5											
Rear Abut	698.210	4.5		697.127			9.705	**690.818**		-0.46%	
Midspan	697.453		-2.772	696.579	1.98	1.32			138.070	-1.13%	-0.67%
Forward Abut	696.695	5		695.570		**OK!**	9.83	**689.251**		-1.80%	



Calculations For	Innerbelt Bridge No. 6	Job No.	49633	Sheet No.
Made by	AKS	Date	4/18/2011	1/10/2012
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HNTB

Title: Rotation Calculation, no DL Deflection - Beams 6 thru 10

Units ("in" or "mm")	in	
Slab Thickness	8.50 in	*CAMBER: (-)=Upward, (+)=Downward
Beam Depth	66.00 in	
CL Bm to Top Flange Edge	24.00 in	(See Sketch Below for Clarity of Values)

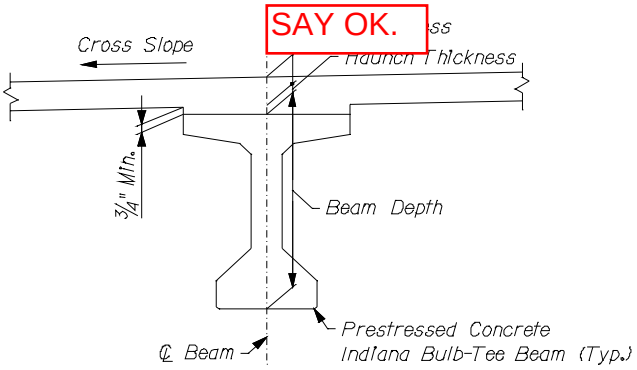
Mid-Span Non-Comp. & Composite DL Deflections

Span 1
0.000 in

These values are added into Mid-Span Elevations

Location	Top Deck Elevation (ft)	Haunch at Ends (in)	Initial Camber* (in)	Top Beam Elevation (ft)	Net Haunch (in)	Min. Allow Haunch (in)	Bearing Height (in)	Bm Seat Elevation (ft)	Span Length (ft)	Beam Slope (%)	Camber Slope (%)
Span 1 - WB Beam 6											
Rear Abut	697.840	4.5		696.757			9.705	**690.448**		-0.33%	
Midspan	697.153		-2.772	696.299	1.75	1.27			138.070	-1.00%	-0.67%
Forward Abut	696.504	5		695.379		**OK!**	9.83	**689.060**		-1.67%	
Span 1 - WB Beam 7											
Rear Abut	697.470	4.5		696.387			9.705	**690.078**		-0.20%	
Midspan	696.864		-2.772	696.017	1.67	1.27			138.070	-0.87%	-0.67%
Forward Abut	696.310	5		695.185		**OK!**	9.83	**688.866**		-1.54%	
Span 1 - WB Beam 8											
Rear Abut	697.100	4.5		696.017			9.705	**689.708**		-0.06%	
Midspan	696.574		-2.772	695.747	1.43	1.27			138.070	-0.73%	-0.67%
Forward Abut	696.140	5		695.015		**OK!**	9.83	**688.696**		-1.39%	
Span 1 - WB Beam 9											
Rear Abut	696.730	4.5		695.647			9.705	**689.338**		0.09%	
Midspan	696.290		-2.772	695.475	1.28	1.23			138.070	-0.58%	-0.67%
Forward Abut	695.967	5		694.842		**OK!**	9.83	**688.523**		-1.25%	
Span 1 - WB Beam 10											
Rear Abut	696.360	4.5		695.277			9.705	**688.968**		0.25%	
Midspan	696.015		-2.828	695.213	1.13	1.24			138.070	-0.43%	-0.68%
Forward Abut	695.802	5		694.677	**NEED MORE HAUNCH!**		9.83	**688.358**		-1.12%	

SAY OK.



Calculations For	Innerbelt Bridge No. 6	Job No.	49633	Sheet No.
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HNTB

TITLE: Actual Slope Results from Geometry Calculations

Actual Slope From Geometry Calculations (Beam Slope Final Case, no DL Deflections)

Span	1	
Girder	Left (Rear)	Right (Fwd.)
1	-0.86%	-2.23%
2	-0.71%	-2.06%
3	-0.60%	-1.94%
4	-0.58%	-1.92%
5	-0.46%	-1.80%
6	-0.33%	-1.67%
7	-0.20%	-1.54%
8	-0.06%	-1.39%
9	0.09%	-1.25%
10	0.28%	-1.09%
Max	-0.86%	-2.23%

Calculations For	Innerbelt Bridge No. 6	Job No.	49633	Sheet No.
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HNTB

Title: CONSPAN Reaction Output for Bearing Design

CONSPAN values are opposite in the Haunch & Beam Seats Spreadsheet

1.15 Impact Factor (Assume truck loading and lane loading each contribute half, and impact not applied to lane)

	Left Support (Rear Abutment)							Left Support (Rear Abutment)		
	Self Wt.	Deck+Haunch	Diaphragm	DL-Prec.(DC)	**DC1**	Live	**LL**	DC(Max)	DW(Max)	**DC2+DW**
Span 1										
Beam 1	69.6	52.2	0.4	23.3	**145.5**	110.5	**96.09**	8.8	32.5	**41.3**
Beam 2	69.6	57.2	0.8	18.0	**145.6**	121.4	**105.57**	9.6	35.5	**45.1**
MAX:					**145.6**		**105.57**			**45.1**
MIN:					**145.5**		**96.09**			**41.3**

	Right Support (Forward Abutment)							Right Support (Forward Abutment)		
	Self Wt.	Deck+Haunch	Diaphragm	DL-Prec.(DC)	**DC1**	Live	**LL**	DC(Max)	DW(Max)	**DC2+DW**
Span 1										
Beam 1	69.6	52.2	0.4	23.3	**145.5**	110.5	**96.09**	8.8	32.5	**41.3**
Beam 2	69.6	57.2	0.8	18.0	**145.6**	121.4	**105.57**	9.6	35.5	**45.1**
MAX:					**145.6**		**105.57**			**45.1**
MIN:					**145.5**		**96.09**			**41.3**

	Span 1	
MAX	Rear Abut.	Fwd. Abut.
DC1	145.6	145.6
DC2+DW	45.1	45.1
LL	105.57	105.57
MIN		
DC1	145.5	145.5
DC2+DW	41.3	41.3
LL	96.09	96.09

Live Load Deflection at Midspan (from CONSPAN output)

	Live Load
Span 1	
Beam 1	-1.195 in.
Beam 2	-1.022 in.
MAX:	-1.218 in.

Calculations For	Innerbelt Bridge No. 6	Job No.	49633	Sheet No.
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TITLE: Bearing Pad Design - Rear Abutment

ELASTOMERIC BEARING PAD DESIGN USING AASHTO LRFD SECTION 14.7.6.3 - METHOD A

	Max	Min	
DC1 Reaction	145.60 kips	145.50 kips	From Beam/Girder Output
DC2 + DW Reaction	45.10 kips	41.30 kips	
LL Reaction (No Impact)	105.57 kips	96.09 kips	
Total Unfactored DL + LL Reaction, R_{Total} =	296.27 kips	282.89 kips	

Span Length, L_{span} =	138.07 ft	
Temperature Deflection, Δ_T =	0.298 in	(Temp Rise/Fall x Coeff. Therm Exp/Contr x Length x Load Factor = 60°F x 0.000006 x (L_{span}/2) x 12 in/ft x 1.0 (γ_{TG}))
Shrinkage/Creep Deflection, Δ_C =	0.249 in	(Shrinkage/Creep Coeff. x Length = (0.0005-0.0002) x (L_{span}/2) x (12 in./ft)
Min. Req'd Elast. Thick., $t = 2(\Delta_T + \Delta_C)$ =	1.094 in	
Pad Length (along beam), L =	12.000 in	
Pad Width (across beam), W =	20.000 in	
Durometer Hardness =	50	
For Bearing Stress Check: Shear Modulus, G =	95 psi	Based on Durometer Hardness, See Table 14.7.6.2-1
For Rotation Check: Shear Modulus, G =	130 psi	
Bearing Area, A =	240.00 in^2	
Bearing Perimeter =	64.000 in	
Number of External Layers =	2	
External Layer Thickness, h_{re} =	0.2876 in	
Shim Thickness =	0.0747 in	
Number of Internal Layers =	3	
Internal Layer Thickness, h_{ri} =	0.4229 in	
Shim Thickness =	0.0747 in	
Number of Layers, n =	4	If h_{re} is ≥ h_{ri}/2 then n may be increased by 1/2 of each such layer
Shape Factor, $S = L*W / 2*h_{ri}*(L + W)$ =	8.87	Eq'n 14.7.5.1-1
Is $S^2 / n < 22$ per 14.7.6.1?	**Yes!**	If the answer is "No!", Method B must be used for design
Shear Deformation Prevented, Yes or No?	No	
Allowable Bearing Stress, σ_s =	1250.0 psi	Eq'n 14.7.6.3.2-6 & Eq'n 14.7.6.3.2-7

Maximum Bearing Stress Check

Actual Bearing Stress, R_{Total} / A =	1234.4 psi	**< Allowable…OK!**
Total Elastomeric Thickness, t =	1.844 in	**> Min. t…OK!**
Total Bearing Thickness =	**2.143 in**	
Minimum Req'd L (or W) for Stability =	6.428 in	**< L (or W)…OK!**
Shear Stiffness, K = G A / t =	12.37 kips/in	
Min. Rxn to Prevent Slip = F / μ = K Δ / 0.2 =	18.44 kips	**< Total DL Rxn…OK!**

Initial Compressive Deflection Check (This check only necessary if there is a deck joint) ** NO DECK JOINT **

	DL Only	LL Only	Total	
Compressive Stress =	778.3 psi	400.4 psi	1178.7 psi	
~~Compressive Strain, ε_c =~~	~~0.0127~~	~~0.0065~~	~~0.0201~~	~~From Eqn C14.7.5.3.6-1 or Figure C14.7.6.3.3-1~~
~~Compressive Defl., $\delta_c = \varepsilon_c t$ =~~	~~0.023 in~~	~~0.012 in~~	~~0.037 in~~	~~Eqn 14.7.5.3.6-1 & Eqn 14.7.5.3.6-2~~
~~Max. Beam Slope Prior to Slab Pour =~~	~~#REF!~~	~~Sum of Elevation Change and Camber Slope (Max all beams)~~		
~~Bevel =~~	~~-0.86%~~			
~~Actual Slope =~~	~~#REF!~~			
~~Initial Allow. Slope, $\alpha = 2\Delta_c / L$ =~~	~~0.390%~~	~~**#REF!**~~		← Ignore this check if bridge is jointless

Rotation Check

Step 1: Build profile grade into a tapered sole plate or concrete pedestal so that bearing does not have to be designed for that rotation.

Slope Due to Profile Grade, θ_G =	-0.0086 rad	+ *CCW*	From Geometry Haunch Calc.'s - Max All Beams - Instantaneous Slope at Abut.
Sole Plate MinimumThickness =	1.500 in		
Add'l Length of Sole Plate Beyond Bearing Pad =	0.500 in	(This value is applied to both sides)	
Sole Plate Maximum Thickness =	1.625 in	← Based on length of pad + add'l plate length either side, set thickness to the nearest 1/8"	

Step 2: Determine the total rotation.

Rotation for Uncertainties, θ_U =	0.0050 rad	+ *CCW*	AASHTO LRFD 14.4.2.1
Live Load Deflection, Δ_{LL} =	-1.218 in	From Beam/Girder Output	
LL Rotation, $\theta_{LL} = 4 * \Delta_{LL} / L_{span}$ =	-0.0029 rad	(or max. from girder output)	
Total Rotation, $\theta_{total} = \theta_U + \theta_{LL}$ =	0.0021 rad		

Step 3: The limits for combined compression and rotation must be satisfied to ensure that no uplift will occur.

Allowbale Stress $\sigma_S \geq 0.5*G*S*(L/h_{ri})^2*(\theta_{total}/n)$ =	239.0 psi	**< Allowable Stress…OK!**

Calculations For	Innerbelt Bridge No. 6	Job No.	49633	Sheet No.
Made by	AKS	Date	4/18/2011	
Checked by	DBT	Date	4/19/2011	
Backchecked by	AKS	Date	4/19/2011	

TITLE: HP Post Design - Rear Abutment

Shape =	HP 10x57
A_s =	16.8 in2
Height, l =	4.5 in.
r_x =	4.18 in.

DC1 Reaction =	145.60 kips	γ_{DC} =	1.25
DC2 + DW Reaction =	45.10 kips	γ_{DW} =	1.5
LL Reaction (No Impact) =	105.57 kips	γ_{LL} =	1.75

Max. Total Factored Reaction, R = 434.39 kips

Compression Check

K =	0.65	AASHTO 4.6.2.5		
F_y =	50 ksi			
E =	29000 ksi			
$\lambda = (Kl / r_x \pi)^2 * (F_y / E)$ =	0.00	<	2.25	Use AASHTO Eq'n 6.9.4.1-1

Nominal Comp. Resistance = $P_n = 0.66^{\lambda} F_y A_s$ =	839.97 kips				
ϕ_c =	0.9	AASHTO 6.5.4.2			
$P_r = \phi_c P_n$ =	755.97 kips	>	434.39 kips	OK	AASHTO 6.9.2.1

Bearing Check (AISC J7, 13th Ed.)

Actual Bearing Stress = R / A_s = 25.9 ksi

ϕ =	0.75			
Allowable Bearing Stress = ϕ 0.9 F_y =	33.8 ksi	>	25.9 ksi	OK

Rear Abutment Bearing Summary

Average Top Plate Thickness =	1.563 in.	
HP 10x57 Post Height =	4.500 in.	
Elastomer Thickness =	2.143 in.	
Bottom Load Plate Thickness =	1.500 in.	
Total Bearing Height =	9.705 in.	(enter back into Beam Seat Calcs)

Calculations For	Innerbelt Bridge No. 6	Job No.	49633	Sheet No.
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HNTB

TITLE: Bearing Pad Design - Forward Abutment

ELASTOMERIC BEARING PAD DESIGN USING AASHTO LRFD SECTION 14.7.6.3 - METHOD A

	Max	Min	
DC1 Reaction	145.60 kips	145.50 kips	From Beam/Girder Output
DC2 + DW Reaction	45.10 kips	41.30 kips	
LL Reaction (No Impact)	105.57 kips	96.09 kips	
Total Unfactored DL + LL Reaction, R_{Total} =	296.27 kips	282.89 kips	

Span Length, L_{span} =	138.07 ft	
Temperature Deflection, Δ_T =	0.298 in	(Temp Rise/Fall x Coeff. Therm Exp/Contr x Length x Load Factor = 60°F x 0.000006 x (L_{span}/2) x 12 in/ft x 1.0 (γ_{TG}))
Shrinkage/Creep Deflection, Δ_C =	0.249 in	(Shrinkage/Creep Coeff. x Length = (0.0005-0.0002) x (L_{span}/2) x (12 in./ft)
Min. Req'd Elast. Thick., t = 2 Δ =	1.094 in	
Pad Length (along beam), L =	12.000 in	
Pad Width (across beam), W =	20.000 in	
Durometer Hardness =	50	
For Bearing Stress Check: Shear Modulus, G =	95 psi	Based on Durometer Hardness, See Table 14.7.6.2-1
For Rotation Check: Shear Modulus, G =	130 psi	
Bearing Area, A =	240.00 in^2	
Bearing Perimeter =	64.000 in	
Number of External Layers =	2	
External Layer Thickness, h_{re} =	0.2876 in	
Shim Thickness =	0.0747 in	
Number of Internal Layers =	3	
Internal Layer Thickness, h_{ri} =	0.4229 in	
Shim Thickness =	0.0747 in	
Number of Layers, n =	4	If h_{re} is ≥ h_{ri}/2 then n may be increased by 1/2 of each such layer
Shape Factor, S = L*W / 2*h_{ri}*(L + W) =	8.87	Eq'n 14.7.5.1-1
Is S^2 / n < 22 per 14.7.6.1?	**Yes!**	If the answer is "No!", Method B must be used for design
Shear Deformation Prevented, Yes or No?	No	
Allowable Bearing Stress, σ_s =	1250.0 psi	Eq'n 14.7.6.3.2-6 & Eq'n 14.7.6.3.2-7

Maximum Bearing Stress Check

Actual Bearing Stress, R_{Total} / A =	1234.4 psi	**< Allowable…OK!**

Total Elastomeric Thickness, t =	1.844 in	**> Min. t…OK!**
Total Bearing Thickness =	**2.143 in**	
Minimum Req'd L (or W) for Stability =	6.428 in	**< L (or W)…OK!**

Shear Stiffness, K = G A / t =	12.37 kips/in	
Min. Rxn to Prevent Slip = F / μ = K Δ / 0.2 =	18.44 kips	**< Total DL Rxn…OK!**

Initial Compressive Deflection Check (This check only necessary if there is a deck joint) ** NO DECK JOINT **

	DL Only	LL Only	Total	
Compressive Stress =	778.3 psi	400.4 psi	1178.7 psi	
~~Compressive Strain, ε_c =~~	~~0.0127~~	~~0.0065~~	~~0.0201~~	~~From Eqn C14.7.5.3.6-1 or Figure C14.7.6.3.3-1~~
~~Compressive Defl., $\delta_d = \varepsilon_c$ t =~~	~~0.023 in~~	~~0.012 in~~	~~0.037 in~~	~~Eqn 14.7.5.3.6-1 & Eqn 14.7.5.3.6-2~~
~~Max. Beam Slope Prior to Slab Pour =~~	~~#REF!~~	~~Sum of Elevation Change and Camber Slope (Max all beams)~~		
~~Bevel =~~	~~2.23%~~			
~~Actual Slope =~~	~~#REF!~~			
~~Initial Allow. Slope, $\alpha = 2\Delta_c / L$ =~~	~~0.390%~~	~~#REF!~~		← Ignore this check if bridge is jointless

Rotation Check

Step 1: Build profile grade into a tapered sole plate or concrete pedestal so that bearing does not have to be designed for that rotation.

Slope Due to Profile Grade, θ_G =	0.0223 rad	+ *CW*	From Geometry Haunch Calc.'s - Max All Beams - Instantaneous Slope at Abut.
Sole Plate MinimumThickness =	1.500 in		
Add'l Length of Sole Plate Beyond Bearing Pad =	0.500 in	(This value is applied to both sides)	
Sole Plate Maximum Thickness =	1.875 in	← Based on length of pad + add'l plate length either side, set thickness to the nearest 1/8"	

Step 2: Determine the total rotation.

Rotation for Uncertainties, θ_U =	0.0050 rad	+ *CW*	AASHTO LRFD 14.4.2.1
Live Load Deflection, Δ_{LL} =	-1.218 in	From Beam/Girder Output	
LL Rotation, $\theta_{LL} = 4 * \Delta_{LL} / L_{span}$ =	-0.0029 rad	(or max. from girder output)	
Total Rotation, $\theta_{total} = \theta_U + \theta_{LL}$ =	0.0021 rad		

Step 3: The limits for combined compression and rotation must be satisfied to ensure that no uplift will occur.

Allowbale Stress $\sigma_S \geq 0.5*G*S*(L/h_{ri})^{2}*(\theta_{total}/n)$ =	239.0 psi	**< Allowable Stress…OK!**

Calculations For	Innerbelt Bridge No. 6	Job No.	49633	Sheet No.
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HNTB

TITLE: HP Post Design - Forward Abutment

Shape =	HP 10x57
A_s =	16.8 in2
Height, l =	4.5 in.
r_x =	4.18 in.

DC1 Reaction =	145.60 kips	γ_{DC} =	1.25
DC2 + DW Reaction =	45.10 kips	γ_{DW} =	1.5
LL Reaction (No Impact) =	105.57 kips	γ_{LL} =	1.75

Max. Total Factored Reaction, R = 434.39 kips

Compression Check

K =	0.65	AASHTO 4.6.2.5		
F_y =	50 ksi			
E =	29000 ksi			
$\lambda = (Kl / r_x \pi)^2 * (F_y / E)$ =	0.00	<	2.25	Use AASHTO Eq'n 6.9.4.1-1

Nominal Comp. Resistance = $P_n = 0.66^{\lambda} F_y A_s$ =	839.97 kips				
ϕ_c =	0.9	AASHTO 6.5.4.2			
$P_r = \phi_c P_n$ =	755.97 kips	>	434.39 kips	OK	AASHTO 6.9.2.1

Bearing Check (AISC J7, 13th Ed.)

Actual Bearing Stress = R / A_s = 25.9 ksi

ϕ =	0.75			
Allowable Bearing Stress = ϕ 0.9 F_y =	33.8 ksi	>	25.9 ksi	OK

Forward Abutment Bearing Summary

Average Top Plate Thickness =	1.688 in.
HP 10x57 Post Height =	4.500 in.
Elastomer Thickness =	2.143 in.
Bottom Load Plate Thickness =	1.500 in.
Total Bearing Height =	9.830 in. (enter back into Beam Seat Calcs)

Calculations For	Innerbelt Bridge No. 6	Job No.	49633	Sheet No.
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HNTB

TITLE: Average Compressive Strain; From Fig. C14.7.5.3.3-1

Substructure	Condition	DL Only	Total
1	Stress (psi)	778	1179
	Strain (%)	3.51	5.05
2	Stress (psi)	778	1179
	Strain (%)	3.51	5.05

Calculations For	Innerbelt Bridge No. 6	Job No.	49633	Sheet No.
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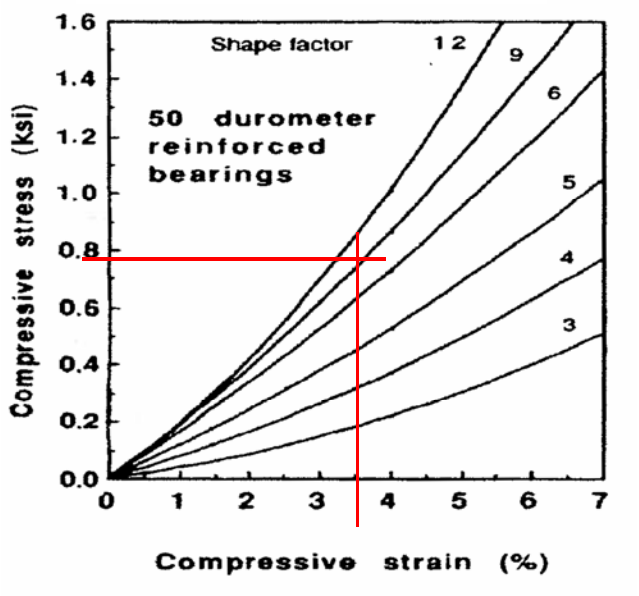
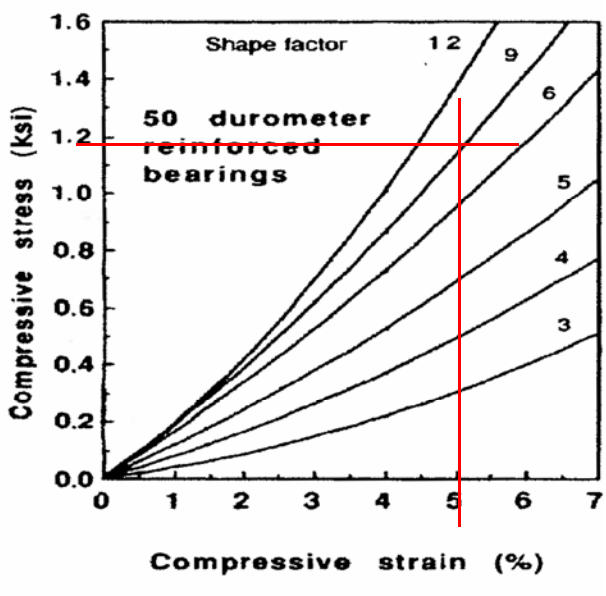
HNTB

TITLE:	Compressive Strain Calc. - Rear Abutment		
DL Only:	778 psi	Shape Factor:	8.87
Total:	1179 psi	Bearing Check Type:	Rotation





Dead Load

DL Only
50 Durometer Strain = 3.51

Dead + Live Load

DL + LL
50 Durometer Strain = 5.05

Calculations For	Innerbelt Bridge No. 6	Job No.	49633	Sheet No.
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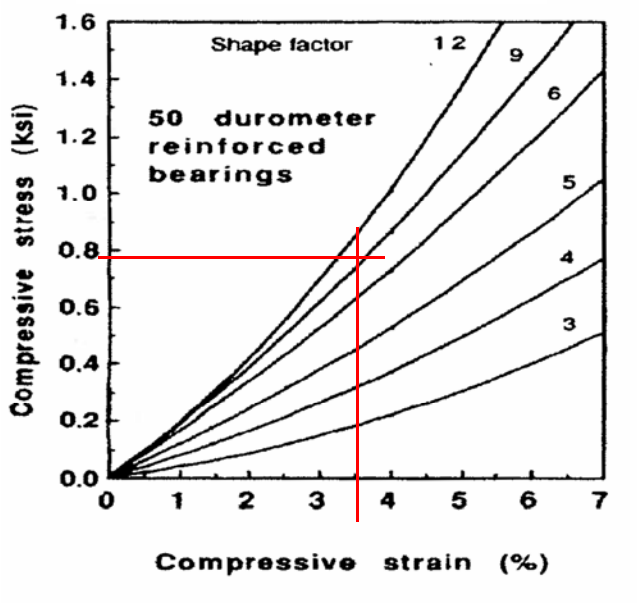
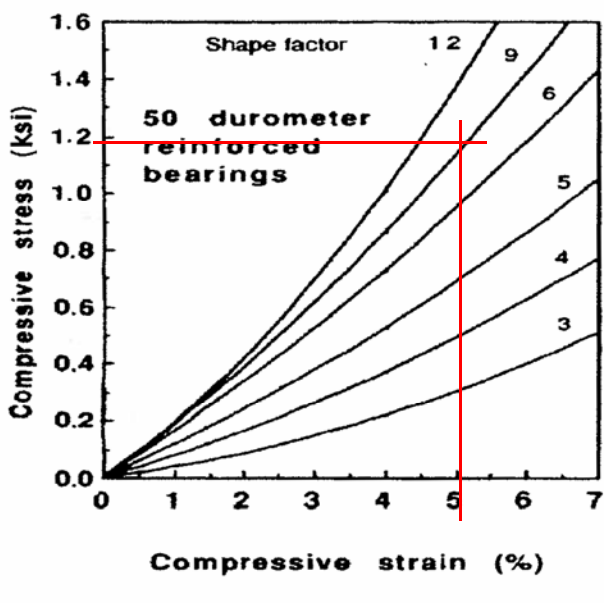
Backchecked by	AKS	Date	4/19/2011	

HNTB

TITLE: Compressive Strain Calc. - Forward Abutment

DL Only:	778 psi	Shape Factor:	8.87
Total:	1179 psi	Bearing Check Type:	Rotation





Dead Load

DL Only
50 Durometer Strain = 3.51

Dead + Live Load

DL + LL
50 Durometer Strain = 5.05

Rel. Humid.(RH)	75.0	%
Es	28500.0	ksi
Eci	5250	ksi

AASHTO LOSSES

Elastic Shortening 22.18 ksi (Eq 5.9.5.2.3a-1), (fcgp= 4.086 ksi)

	Elastic Gains	Gains		Adjustment	
due	to Precast Loads	-6.97	ksi	0.81	ksi
due	to Composite Loads	-3.06	ksi	0.35	ksi
due	to Live Loads	-6.18	ksi	0.89	ksi

Time Dependent Losses (Approximate Method (Art.5.9.5.3))

		Initial		Final		
Steel relaxation	0.00	ksi		2.40	ksi	(Eq 5.9.5.3-1)
Concrete shrinkage	0.00	ksi		6.71	ksi	(Eq 5.9.5.3-1)
Concrete creep	0.00	ksi		15.22	ksi	(Eq 5.9.5.3-1)
Sub-total	22.18	ksi	(10.95 %)	10.17	ksi	(5.02 %)
Total Prestress Losses				32.34	ksi	(15.97 %)

Prestressing Stress Limit Check (Table 5.9.3.1)
initial fpi = 202.5 ksi < 0.75 fpu, OK
initial fpe = 170.2 ksi < 0.80 fpy, OK

SHEAR/MOMENT ENVELOPE (&REACTIONS)

SHEAR AND MOMENT ENVELOPE : Span : 1, Beam : 1, SERVICE I
Shears: kips, Moments: kft

		Bearing	Trans	H/2	0.10L	0.20L	0.30L	0.40L	Midspan
Location,	ft	0.00	2.17	3.06	13.14	27.11	41.09	55.06	69.04
Self wt. :	M	0.0	148.3	208.5	827.6	1516.7	2009.0	2304.3	2402.8
(Max)	V	69.6	67.4	66.5	56.4	42.3	28.2	14.1	0.0
DL-Prec. :	M	0.0	49.7	69.9	277.3	508.3	673.2	772.2	805.2
DC(Max)	V	23.3	22.6	22.3	18.9	14.2	9.4	4.7	0.0
DL-Prec. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DW(Max)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Deck + :	M	0.0	111.3	156.4	621.0	1138.2	1507.6	1729.2	1803.1
Haunch (Max)	V	52.2	50.6	49.9	42.3	31.7	21.1	10.6	0.0
Diaphragm :	M	0.0	0.9	1.2	5.3	11.0	14.9	16.8	18.6
(Max)	V	0.4	0.4	0.4	0.4	0.4	0.1	0.1	0.1
DL-Comp :	M	-0.0	18.7	26.3	104.2	191.0	253.0	290.2	302.6
DC(Max)	V	8.8	8.5	8.4	7.1	5.3	3.5	1.8	0.0
DL-Comp :	M	-0.0	69.2	97.2	385.8	707.1	936.6	1074.3	1120.2
DW(Max)	V	32.5	31.4	31.0	26.3	19.7	13.1	6.6	0.0
LL + I :	M+	-0.0	228.4	320.8	1269.3	2312.5	3037.7	3466.9	3590.2
	V	110.5	108.1	107.1	95.8	80.5	29.9	8.4	28.2
LL + I :	M-	-0.0	-0.0	-0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
LL + I :	Vmx	110.5	108.1	107.1	95.9	81.7	68.2	55.4	43.4
	M	-0.0	225.2	315.9	1224.9	2154.6	2724.5	2965.9	2916.1
Total :	M+	0.0	626.5	880.2	3490.6	6384.8	8432.0	9653.9	10042.7
	V	297.3	289.0	285.6	247.2	194.1	105.5	46.2	28.3
Total :	M-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total :	Vmx	297.3	289.0	285.6	247.2	195.3	143.8	93.2	43.6
	M	0.0	623.3	875.3	3446.2	6226.9	8118.8	9152.9	9368.6

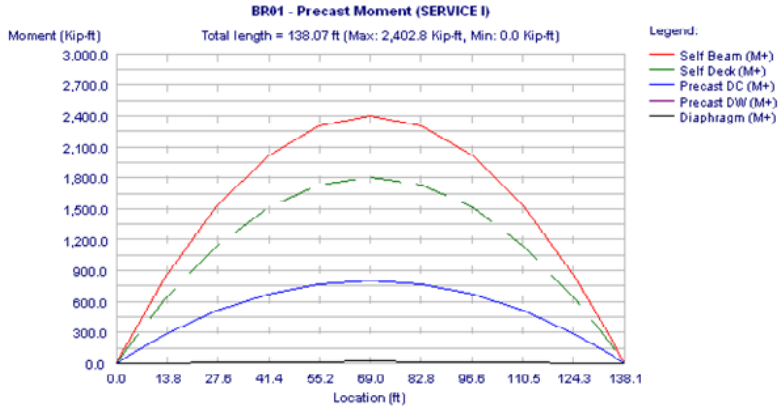
		0.60L	0.70L	0.80L	0.90L	H/2	Trans	Bearing
Location,	ft	83.01	96.98	110.96	124.93	135.01	135.91	138.07
Self wt. :	M	2304.3	2009.0	1516.7	827.6	208.5	148.3	0.0
(Max)	V	14.1	28.2	42.3	56.4	66.5	67.4	69.6
DL-Prec. :	M	772.2	673.2	508.3	277.3	69.9	49.7	0.0
DC(Max)	V	4.7	9.4	14.2	18.9	22.3	22.6	23.3
DL-Prec. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DW(Max)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Deck + :	M	1729.2	1507.6	1138.2	621.0	156.4	111.3	0.0
Haunch (Max)	V	10.6	21.1	31.7	42.3	49.9	50.6	52.2
Diaphragm :	M	16.8	14.9	11.0	5.3	1.2	0.9	-0.0
(Max)	V	0.1	0.1	0.4	0.4	0.4	0.4	0.4
DL-Comp :	M	290.2	253.0	191.0	104.2	26.3	18.7	-0.0
DC(Max)	V	1.8	3.5	5.3	7.1	8.4	8.5	8.8
DL-Comp :	M	1074.3	936.6	707.1	385.8	97.2	69.2	0.0
DW(Max)	V	6.6	13.1	19.7	26.3	31.0	31.4	32.5

		0.60L	0.70L	0.80L	0.90L	H/2	Trans	Bearing
LL + I :	M+	3466.9	3037.7	2312.5	1269.3	320.8	228.4	-0.0
	V	8.4	29.9	80.5	95.8	107.1	108.1	110.5
LL + I :	M-	0.0	0.0	0.0	0.0	-0.0	-0.0	-0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
LL + I :	Vmx	55.4	68.2	81.7	95.9	107.1	108.1	110.5
	M	2965.9	2724.5	2154.6	1224.9	315.9	225.2	0.0
Total :	M+	9653.9	8432.0	6384.8	3490.6	880.2	626.5	0.0
	V	46.2	105.5	194.1	247.2	285.6	289.0	297.3
Total :	M-	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total :	Vmx	93.2	143.8	195.3	247.2	285.6	289.0	297.3
	M	9152.9	8118.8	6226.9	3446.2	875.3	623.3	0.0

REACTIONS (kips), SERVICE I

Load Type	Left Support	Right Support
Self Wt.	69.6	69.6
Deck+Haunch	52.2	52.2
Diaphragm	0.4	0.4
DL-Prec.(DC)	23.3	23.3
DL-Prec.(DW)	0.0	0.0
DL-Comp.(DC)	93.7	93.7
DL-Comp.(DW)	347.1	347.1
Live	111.3	111.3
Pedestrian	0.0	0.0

Upward reactions are positive.
Live Load reactions are per lane with no distribution factor and no impact.
Non-composite load types are per beam.
Composite and Pedestrian load types are per total bridge width.



BR01 - Precast Moment, Span 1, Beam 1, SERVICE I

DESIGN SUMMARY

Span: 1, Beam: 1, Exterior beam

Beam type:	I-Girder,	ODOT 66" - 4ft - SHOP
Precast Length,	ft	139.74
Release Length,	ft	139.74
Strand Pattern:	Straight/Draped	Depr. Point: 0.45 L
Strand:	6/10-270K-LL	
Strand Es,	ksi:	28500.0
No. of strands:	60	
	Draped:	17
	Straight:	43
Concrete Strength:		
	f'ci:	7.5 ksi
	f'c:	11.0 ksi
	f'ct:	4.5 ksi
Initial losses:	10.95 %	
Final losses:	15.97 %	

Specification	Allowable	Computed	Location	Status
Release Stresses (ksi) (Art. 5.9.4.1)				
Precast Bot (compression)	4.500	4.472	Depress	OK
Precast Top w/ no reinf. (tension)	-0.200	0.422	Midspan	
Precast Top w/ reinf. (tension)	-0.657			
Strength I (Art. 3.4.1, 5.7.3.1.1)	Provided	Required	Location	Status
Ult. Moment (k.ft)	18062.68	14628.48	Midspan	OK
Debonding Limits (Art. 5.11.4.3)	Allowable	Computed		Status
Max. Debond per Row	40.00 %	0.00 %		OK
Max. Debond Total	25.00 %	0.00 %		OK

Positive Moment Envelope Stresses (ksi) (Art. 3.4.1 and 5.9.4.2)

Specification	Allow	Final 1 Comp	Loc.	Allow	Final 2 Comp	Loc.	Allow	Final 3 Comp	Loc.
Service I Limit State - Compressive	Stresses	Only							
Precast Top	6.600	3.698	0.4L/0.6L	4.950	2.792	0.4L/0.6L			
Precast Bot	6.600	3.636	Transfer	4.950	3.764	Transfer			
Service III Limit State - Tensile	Stresses	Only							
Precast Top	-0.314	0.174	Bearing						
Precast Bot	-0.314	-0.209	Midspan						
Fatigue I Limit State - Compressive	Stresses	Only							
Precast Top							4.400	1.980	0.4L/0.6L
Precast Bot							4.400	1.796	Transfer

CAMBER / DEFLECTION: (PCI Design Handbook - 4th Ed.- Table 4.6.2)
0.5 x L = 69.04 ft

	Release	Mult	Erection	Mult	Final
Prestress	6.607	1.80	11.893	2.20	14.536
Self Wt.	-3.050	1.85	-5.643	2.40	-7.321
Deck + Haunch			-1.801	2.30	-4.143
DL-Prec. (DC)			-0.804	3.00	-2.413
Diaphragm			-0.018	3.00	-0.053
DL-Prec. (DW)			0.000	3.00	0.000
DL-Comp. (DC)			-0.170	3.00	-0.509
DL-Comp. (DW)			-0.629	3.00	-1.886
Live Load					-1.157
Total	3.557		2.829		-2.946

Positive values indicate upward deflection.

AASHTO LOSSES

Elastic Shortening 22.18 ksi (Eq 5.9.5.2.3a-1), (fcgp= 4.086 ksi)

Elastic Gains		Gains		Adjustment	
due	to Precast Loads	-6.98	ksi	0.81	ksi
due	to Composite Loads	-3.33	ksi	0.38	ksi
due	to Live Loads	-5.41	ksi	0.78	ksi

Time Dependent Losses (Approximate Method (Art.5.9.5.3))

		Initial		Final		
Steel relaxation	0.00	ksi		2.40	ksi	(Eq 5.9.5.3-1)
Concrete shrinkage	0.00	ksi		6.71	ksi	(Eq 5.9.5.3-1)
Concrete creep	0.00	ksi		15.22	ksi	(Eq 5.9.5.3-1)
Sub-total	22.18	ksi	(10.95 %)	10.58	ksi	(5.22 %)
Total Prestress Losses				32.76	ksi	(16.18 %)

Prestressing Stress Limit Check (Table 5.9.3.1)
initial fpi = 202.5 ksi < 0.75 fpu, OK
initial fpe = 169.7 ksi < 0.80 fpy, OK

SHEAR/MOMENT ENVELOPE (&REACTIONS)

SHEAR AND MOMENT ENVELOPE : Span : 1, Beam : 2, SERVICE I
Shears: kips, Moments: kft

		Bearing	Trans	H/2	0.10L	0.20L	0.30L	0.40L	Midspan
Location,	ft	0.00	2.17	3.06	13.14	27.11	41.09	55.06	69.04
Self wt. :	M	0.0	148.3	208.5	827.6	1516.7	2009.0	2304.3	2402.8
(Max)	V	69.6	67.4	66.5	56.4	42.3	28.2	14.1	0.0
DL-Prec. :	M	0.0	38.3	53.8	213.7	391.7	518.8	595.1	620.5
DC(Max)	V	18.0	17.4	17.2	14.6	10.9	7.3	3.6	0.0
DL-Prec. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DW(Max)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Deck + :	M	0.0	121.8	171.2	679.7	1245.7	1649.9	1892.5	1973.3
Haunch (Max)	V	57.2	55.4	54.6	46.3	34.7	23.1	11.6	0.0
Diaphragm :	M	0.0	1.8	2.5	10.8	22.4	30.3	34.1	38.0
(Max)	V	0.8	0.8	0.8	0.8	0.8	0.3	0.3	0.3
DL-Comp :	M	-0.0	20.4	28.7	114.1	209.0	276.9	317.6	331.1
DC(Max)	V	9.6	9.3	9.2	7.8	5.8	3.9	1.9	0.0
DL-Comp :	M	-0.0	75.7	106.4	422.3	773.9	1025.1	1175.8	1226.0
DW(Max)	V	35.5	34.4	33.9	28.8	21.6	14.4	7.2	0.0
LL + I :	M+	-0.0	201.5	283.1	1119.9	2040.3	2680.1	3058.8	3167.5
	V	121.4	118.7	117.6	105.3	88.4	32.9	9.2	30.9
LL + I :	M-	-0.0	-0.0	-0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
LL + I :	Vmx	121.4	118.7	117.6	105.3	89.7	74.9	60.8	47.7
	M	-0.0	198.7	278.7	1080.7	1901.0	2403.8	2616.8	2572.8
Total :	M+	0.0	607.9	854.2	3388.0	6199.7	8190.1	9378.2	9759.3
	V	312.1	303.5	299.9	259.8	204.6	110.0	47.9	31.2
Total :	M-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total :	Vmx	312.1	303.5	299.9	259.8	205.9	152.0	99.5	48.0
	M	0.0	605.1	849.8	3348.9	6060.4	7913.8	8936.2	9164.6

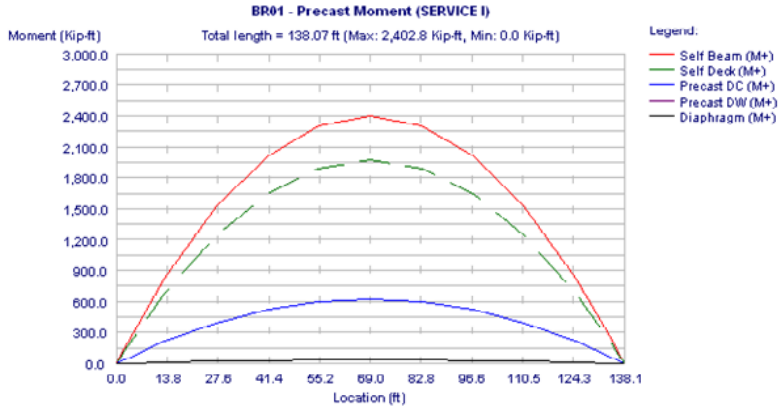
		0.60L	0.70L	0.80L	0.90L	H/2	Trans	Bearing
Location,	ft	83.01	96.98	110.96	124.93	135.01	135.91	138.07
Self wt. :	M	2304.3	2009.0	1516.7	827.6	208.5	148.3	0.0
(Max)	V	14.1	28.2	42.3	56.4	66.5	67.4	69.6
DL-Prec. :	M	595.1	518.8	391.7	213.7	53.8	38.3	0.0
DC(Max)	V	3.6	7.3	10.9	14.6	17.2	17.4	18.0
DL-Prec. :	M	0.0	0.0	0.0	0.0	0.0	0.0	0.0
DW(Max)	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Deck + :	M	1892.5	1649.9	1245.7	679.7	171.2	121.8	0.0
Haunch (Max)	V	11.6	23.1	34.7	46.3	54.6	55.4	57.2
Diaphragm :	M	34.1	30.3	22.4	10.8	2.5	1.8	-0.0
(Max)	V	0.3	0.3	0.8	0.8	0.8	0.8	0.8
DL-Comp :	M	317.6	276.9	209.0	114.1	28.7	20.4	-0.0
DC(Max)	V	1.9	3.9	5.8	7.8	9.2	9.3	9.6
DL-Comp :	M	1175.8	1025.1	773.9	422.3	106.4	75.7	0.0
DW(Max)	V	7.2	14.4	21.6	28.8	33.9	34.4	35.5

		0.60L	0.70L	0.80L	0.90L	H/2	Trans	Bearing
LL + I :	M+	3058.8	2680.1	2040.3	1119.9	283.1	201.5	-0.0
	V	9.2	32.9	88.4	105.3	117.6	118.7	121.4
LL + I :	M-	0.0	0.0	0.0	0.0	-0.0	-0.0	-0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
LL + I :	Vmx	60.8	74.9	89.7	105.3	117.6	118.7	121.4
	M	2616.8	2403.8	1901.0	1080.7	278.7	198.7	0.0
Total :	M+	9378.2	8190.1	6199.7	3388.0	854.2	607.9	0.0
	V	47.9	110.0	204.6	259.8	299.9	303.5	312.1
Total :	M-	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total :	Vmx	99.5	152.0	205.9	259.8	299.9	303.5	312.1
	M	8936.2	7913.8	6060.4	3348.9	849.8	605.1	0.0

REACTIONS (kips), SERVICE I

Load Type	Left Support	Right Support
Self Wt.	69.6	69.6
Deck+Haunch	57.2	57.2
Diaphragm	0.8	0.8
DL-Prec.(DC)	18.0	18.0
DL-Prec.(DW)	0.0	0.0
DL-Comp.(DC)	93.7	93.7
DL-Comp.(DW)	347.1	347.1
Live	111.3	111.3
Pedestrian	0.0	0.0

Upward reactions are positive.
Live Load reactions are per lane with no distribution factor and no impact.
Non-composite load types are per beam.
Composite and Pedestrian load types are per total bridge width.



BR01 - Precast Moment, Span 1, Beam 2, SERVICE I

DESIGN SUMMARY

Span: 1, Beam: 2, Interior beam

Beam type:	I-Girder,	ODOT 66" - 4ft - SHOP
Precast Length,	ft	139.74
Release Length,	ft	139.74
Strand Pattern:	Straight/Draped	Depr. Point: 0.45 L
Strand:	6/10-270K-LL	
Strand Es,	ksi:	28500.0
No. of strands:	60	
	Draped:	17
	Straight:	43
Concrete Strength:		
	f'ci:	7.5 ksi
	f'c:	11.0 ksi
	f'ct:	4.5 ksi
Initial losses:	10.95 %	
Final losses:	16.18 %	

Specification	Allowable	Computed	Location	Status
Release Stresses (ksi) (Art. 5.9.4.1)				
Precast Bot (compression)	4.500	4.472	Depress	OK
Precast Top w/ no reinf. (tension)	-0.200	0.422	Midspan	
Precast Top w/ reinf. (tension)	-0.657			
Strength I (Art. 3.4.1, 5.7.3.1.1)	Provided	Required	Location	Status
Ult. Moment (k.ft)	18185.17	14089.45	Midspan	OK
Debonding Limits (Art. 5.11.4.3)	Allowable	Computed		Status
Max. Debond per Row	40.00 %	0.00 %		OK
Max. Debond Total	25.00 %	0.00 %		OK

Positive Moment Envelope Stresses (ksi) (Art. 3.4.1 and 5.9.4.2)

Specification	Allow	Final 1 Comp	Loc.	Allow	Final 2 Comp	Loc.	Allow	Final 3 Comp	Loc.
Service I Limit State - Compressive	Stresses	Only							
Precast Top	6.600	3.558	0.4L/0.6L	4.950	2.807	0.4L/0.6L			
Precast Bot	6.600	3.638	Transfer	4.950	3.750	Transfer			
Service III Limit State - Tensile	Stresses	Only							
Precast Top	-0.314	0.173	Bearing						
Precast Bot	-0.314	-0.088	Midspan						
Fatigue I Limit State - Compressive	Stresses	Only							
Precast Top							4.400	1.729	0.4L/0.6L
Precast Bot							4.400	1.825	Transfer

CAMBER / DEFLECTION: (PCI Design Handbook - 4th Ed.- Table 4.6.2)
0.5 x L = 69.04 ft

	Release	Mult	Erection	Mult	Final
Prestress	6.607	1.80	11.893	2.20	14.536
Self Wt.	-3.050	1.85	-5.643	2.40	-7.321
Deck + Haunch			-1.971	2.30	-4.534
DL-Prec. (DC)			-0.620	3.00	-1.860
Diaphragm			-0.036	3.00	-0.108
DL-Prec. (DW)			0.000	3.00	0.000
DL-Comp. (DC)			-0.181	3.00	-0.543
DL-Comp. (DW)			-0.670	3.00	-2.009
Live Load					-0.994
Total	3.557		2.772		-2.833

Positive values indicate upward deflection.

SCREEDS

Calculations For	Innerbelt - Bridge B1 (CUY90-1627)	Job No.	49633	Sheet No.
Made by	AKS	Date	4/18/2011	
Checked by	DBT	Date	4/19/2011	
Backchecked by	AKS	Date	4/19/2011	

Title: Innerbelt - Bridge B6 (CUY90-1627) Camber and DL Deflection Design

Screeds, Camber and Dead Load Deflection Approach

References:

ODOT BDM LRFD 2007 (effective 04-16-10)

AASHTO LRFD Standard Specifications for Highway Bridges, 4th Ed. 2007

Notes:

Gather station and elevation at points of bearing and tenth-points along beam, for all beams, using InRoads, as well as station and elevation for profile grade, crown, edge of slab and grade breaks.

Calculations for camber and DL deflection automatically performed. Screed, Final Deck Surface Elevation and Top of Haunch tables automatically filled.

Calculations For	Innerbelt - Bridge B1 (CUY90-1627)	Job No.	49633	Sheet No.
Made by	AKS	Date	4/18/2011	
Checked by	DBT	Date	4/19/2011	
Backchecked by	AKS	Date	4/19/2011	

HNTB

Title: InRoads Output - Beam Screed Lines

Note: Elevations for screed lines are from proposed surface ML-I90WB.dtm. Lines with "Ramp" designation are from proposed surface RMP-RampA3.dtm.

Clearance Report

Report Created: 4/18/2011
Time: 6:52pm

Project: Innerbelt I-90
Description:
File Name: pw:\\HNTBW352.HNTB.ORG:DESIGN_BUILD\Documents\Walsh\49633 Central Viaduct Innerbelt WB Bridge\Design Post Award\Engineering\Modeling\Innerbelt I-90.alg
Last Revised: asteffl 4/18/2011 6:52:30 PM
Baseline Alignment: ML-I-90 WB
Input Grid Factor: 1

Note: All units in this report are in feet unless specified otherwise.

Clearance Alignment: Beam 1

Offset Point	Baseline Station	Offset	Offset Point - Northing	Offset Point - Easting	Offset Point - Elevation	Radial Direction
	175+51.25	-41.9263	667166.3209	2192640.676	699.27	S 23°49'34.10" E
	175+64.84	-42.0039	667171.919	2192653.221	699.18	S 23°37'20.22" E
	175+78.43	-42.1304	667177.517	2192665.766	699.07	S 23°25'06.37" E
	175+92.01	-42.3057	667183.115	2192678.312	698.86	S 23°12'52.58" E
	176+05.60	-42.5299	667188.713	2192690.857	698.63	S 23°00'38.87" E

Clearance Alignment: Beam 1 Ramp

Offset Point	Baseline Station	Offset	Offset Point - Northing	Offset Point - Easting	Offset Point - Elevation	Radial Direction
	176+19.18	-42.803	667194.311	2192703.402	698.40	S 22°48'25.25" E
	176+32.76	-43.1249	667199.909	2192715.947	698.15	S 22°36'11.74" E
	176+46.34	-43.4957	667205.5071	2192728.492	697.91	S 22°23'58.36" E
	176+59.91	-43.9152	667211.1051	2192741.037	697.67	S 22°11'45.14" E
	176+73.49	-44.3836	667216.7031	2192753.582	697.43	S 21°59'32.08" E
	a 176+99.11	-44.8803	667222.3011	2192766.127	697.20	S 21°58'31.41" E

Clearance Alignment: Beam 2

Offset Point	Baseline Station	Offset	Offset Point - Northing	Offset Point - Easting	Offset Point - Elevation	Radial Direction
	175+52.94	-33.0998	667158.9333	2192645.797	699.15	S 23°48'03.05" E
	175+66.55	-33.1835	667164.5313	2192658.343	699.04	S 23°35'47.49" E
	175+80.17	-33.3163	667170.1293	2192670.888	698.93	S 23°23'31.96" E
	175+93.79	-33.498	667175.7273	2192683.433	698.73	S 23°11'16.50" E
	176+07.41	-33.7286	667181.3254	2192695.978	698.52	S 22°59'01.11" E

Clearance Alignment: Beam 2 Ramp

Offset Point	Baseline Station	Offset	Offset Point - Northing	Offset Point - Easting	Offset Point - Elevation	Radial Direction
	176+21.02	-34.0083	667186.9234	2192708.523	698.31	S 22°46'45.83" E
	176+34.63	-34.3369	667192.5214	2192721.068	698.11	S 22°34'30.66" E
	176+48.24	-34.7144	667198.1194	2192733.613	697.90	S 22°22'15.62" E
	176+61.85	-35.1409	667203.7174	2192746.158	697.70	S 22°10'00.73" E
	a 176+87.36	-35.6162	667209.3154	2192758.703	697.50	S 21°58'31.41" E
	a 177+01.09	-36.113	667214.9135	2192771.249	697.29	S 21°58'31.41" E

Calculations For	Innerbelt - Bridge B1 (CUY90-1627)	Job No.	49633	Sheet No.
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Clearance Alignment: Beam 3

Offset Point	Baseline Station	Offset	Offset Point Northing	Offset Point Easting	Offset Point Elevation	Radial Direction
	175+54.63	-24.2741	667151.5456	2192650.919	698.79	S 23°46'31.58" E
	175+68.28	-24.364	667157.1436	2192663.464	698.68	S 23°34'14.33" E
	175+81.93	-24.503	667162.7417	2192676.009	698.55	S 23°21'57.12" E
	175+95.58	-24.691	667168.3397	2192688.554	698.38	S 23°09'39.98" E
	176+09.22	-24.9282	667173.9377	2192701.099	698.19	S 22°57'22.92" E

Clearance Alignment: Beam 3 Ramp

Offset Point	Baseline Station	Offset	Offset Point Northing	Offset Point Easting	Offset Point Elevation	Radial Direction
	176+22.87	-25.2144	667179.5357	2192713.644	698.00	S 22°45'05.95" E
	176+36.51	-25.5497	667185.1337	2192726.189	697.82	S 22°32'49.11" E
	176+50.15	-25.9341	667190.7317	2192738.734	697.64	S 22°20'32.40" E
	176+63.79	-26.3675	667196.3298	2192751.28	697.45	S 22°08'15.85" E
	a 176+89.35	-26.8489	667201.9278	2192763.825	697.27	S 21°58'31.41" E
	a 177+03.08	-27.3457	667207.5258	2192776.37	697.09	S 21°58'31.41" E

Clearance Alignment: Beam 4

Offset Point	Baseline Station	Offset	Offset Point Northing	Offset Point Easting	Offset Point Elevation	Radial Direction
	175+55.60	-19.2325	667147.3252	2192653.844	698.58	S 23°45'39.14" E
	175+69.34	-18.9809	667152.6342	2192666.59	698.45	S 23°33'17.26" E
	175+83.07	-18.779	667157.9432	2192679.335	698.31	S 23°20'55.29" E
	175+96.81	-18.6267	667163.2522	2192692.081	698.14	S 23°08'33.25" E
	176+10.55	-18.5241	667168.5612	2192704.826	697.95	S 22°56'11.17" E
	176+24.29	-18.4711	667173.8702	2192717.572	697.76	S 22°43'49.05" E
	176+38.03	-18.4679	667179.1792	2192730.317	697.58	S 22°31'26.92" E
	176+51.77	-18.5143	667184.4882	2192743.063	697.40	S 22°19'04.80" E
	176+65.51	-18.6104	667189.7972	2192755.808	697.22	S 22°06'42.71" E
	a 176+91.18	-18.7533	667195.1062	2192768.553	697.06	S 21°58'31.41" E
	a 177+04.99	-18.9071	667200.4152	2192781.299	696.81	S 21°58'31.41" E

Clearance Alignment: Beam 5

Offset Point	Baseline Station	Offset	Offset Point Northing	Offset Point Easting	Offset Point Elevation	Radial Direction
	175+57.31	-10.3634	667139.9002	2192658.991	698.21	S 23°44'06.54" E
	175+71.08	-10.118	667145.2092	2192671.737	698.08	S 23°31'42.94" E
	175+84.85	-9.9225	667150.5182	2192684.482	697.94	S 23°19'19.25" E
	175+98.62	-9.7767	667155.8272	2192697.228	697.78	S 23°06'55.49" E
	176+12.39	-9.6807	667161.1362	2192709.973	697.62	S 22°54'31.69" E
	176+26.17	-9.6344	667166.4452	2192722.719	697.45	S 22°42'07.86" E
	176+39.94	-9.638	667171.7541	2192735.464	697.29	S 22°29'44.02" E
	176+53.71	-9.6914	667177.0631	2192748.21	697.13	S 22°17'20.19" E
	176+67.48	-9.7945	667182.3721	2192760.955	696.98	S 22°04'56.39" E
	a 176+93.17	-9.9416	667187.6811	2192773.701	696.84	S 21°58'31.41" E
	a 177+06.98	-10.0955	667192.9901	2192786.446	696.70	S 21°58'31.41" E

Clearance Alignment: Beam 6

Offset Point	Baseline Station	Offset	Offset Point Northing	Offset Point Easting	Offset Point Elevation	Radial Direction
	175+59.04	-1.495	667132.4751	2192664.139	697.84	S 23°42'33.51" E
	175+72.84	-1.256	667137.7841	2192676.884	697.71	S 23°30'08.18" E
	175+86.64	-1.0668	667143.0931	2192689.629	697.58	S 23°17'42.76" E
	176+00.44	-0.9275	667148.4021	2192702.375	697.44	S 23°05'17.27" E
	176+14.24	-0.8381	667153.7111	2192715.12	697.29	S 22°52'51.75" E
	176+28.05	-0.7986	667159.0201	2192727.866	697.15	S 22°40'26.19" E
	176+41.85	-0.8091	667164.3291	2192740.611	697.01	S 22°28'00.63" E
	176+55.66	-0.8694	667169.6381	2192753.357	696.88	S 22°15'35.09" E
	176+69.46	-0.9797	667174.9471	2192766.102	696.74	S 22°03'09.57" E
	a 176+95.17	-1.13	667180.2561	2192778.848	696.62	S 21°58'31.41" E
	a 177+08.98	-1.2838	667185.5651	2192791.593	696.50	S 21°58'31.41" E

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Checked by	DBT	Date	4/19/2011	
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Clearance Alignment: Beam 7

Offset Point	Baseline Station	Offset	Offset Point Northing	Offset Point Easting	Offset Point Elevation	Radial Direction
	175+60.77	7.3726	667125.0501	2192669.286	697.47	S 23°41'00.05" E
	175+74.60	7.6053	667130.3591	2192682.031	697.34	S 23°28'32.98" E
	175+88.43	7.7881	667135.6681	2192694.777	697.22	S 23°16'05.82" E
	176+02.27	7.9208	667140.9771	2192707.522	697.11	S 23°03'38.60" E
	176+16.10	8.0035	667146.2861	2192720.267	696.98	S 22°51'11.34" E
	176+29.94	8.0362	667151.5951	2192733.013	696.86	S 22°38'44.06" E
	176+43.78	8.0189	667156.9041	2192745.758	696.75	S 22°26'16.77" E
	176+57.61	7.9515	667162.2131	2192758.504	696.62	S 22°13'49.50" E
	176+71.45	7.8342	667167.5221	2192771.249	696.52	S 22°01'22.27" E
	a 176+97.16	7.6817	667172.8311	2192783.995	696.41	S 21°58'31.41" E
	a 177+10.97	7.5279	667178.14	2192796.74	696.31	S 21°58'31.41" E

Clearance Alignment: Beam 8

Offset Point	Baseline Station	Offset	Offset Point Northing	Offset Point Easting	Offset Point Elevation	Radial Direction
	175+62.51	16.2394	667117.6251	2192674.433	697.10	S 23°39'26.16" E
	175+76.37	16.4658	667122.9341	2192687.178	696.97	S 23°26'57.33" E
	175+90.24	16.6421	667128.2431	2192699.924	696.86	S 23°14'28.43" E
	176+04.10	16.7682	667133.5521	2192712.669	696.78	S 23°01'59.47" E
	176+17.97	16.8442	667138.861	2192725.415	696.67	S 22°49'30.47" E
	176+31.84	16.8701	667144.17	2192738.16	696.57	S 22°37'01.45" E
	176+45.71	16.8459	667149.479	2192750.906	696.49	S 22°24'32.42" E
	176+59.58	16.7715	667154.788	2192763.651	696.38	S 22°12'03.42" E
	176+73.44	16.647	667160.097	2192776.396	696.30	S 21°59'34.46" E
	a 176+99.16	16.4933	667165.406	2192789.142	696.21	S 21°58'31.41" E
	a 177+12.96	16.3395	667170.715	2192801.887	696.14	S 21°58'31.41" E

Clearance Alignment: Beam 9

Offset Point	Baseline Station	Offset	Offset Point Northing	Offset Point Easting	Offset Point Elevation	Radial Direction
	175+64.25	25.1054	667110.2	2192679.58	696.73	S 23°37'51.82" E
	175+78.15	25.3254	667115.509	2192692.325	696.60	S 23°25'21.24" E
	175+92.05	25.4952	667120.818	2192705.071	696.53	S 23°12'50.59" E
	176+05.95	25.6148	667126.127	2192717.816	696.44	S 23°00'19.88" E
	176+19.85	25.6841	667131.436	2192730.562	696.37	S 22°47'49.13" E
	176+33.75	25.7031	667136.745	2192743.307	696.29	S 22°35'18.36" E
	176+47.65	25.6719	667142.054	2192756.053	696.21	S 22°22'47.59" E
	176+61.55	25.5904	667147.363	2192768.798	696.14	S 22°10'16.85" E
	a 176+87.35	25.4588	667152.672	2192781.544	696.08	S 21°58'31.41" E
	a 177+01.15	25.305	667157.981	2192794.289	696.02	S 21°58'31.41" E
	a 177+14.96	25.1512	667163.29	2192807.034	695.97	S 21°58'31.41" E

Clearance Alignment: Beam 10

Offset Point	Baseline Station	Offset	Offset Point Northing	Offset Point Easting	Offset Point Elevation	Radial Direction
	175+66.01	33.9706	667102.775	2192684.727	696.36	S 23°36'17.04" E
	175+79.94	34.1842	667108.084	2192697.472	696.24	S 23°23'44.70" E
	175+93.87	34.3475	667113.393	2192710.218	696.19	S 23°11'12.29" E
	176+07.80	34.4604	667118.702	2192722.963	696.12	S 22°58'39.82" E
	176+21.73	34.5229	667124.011	2192735.709	696.07	S 22°46'07.31" E
	176+35.67	34.5351	667129.32	2192748.454	696.01	S 22°33'34.79" E
	176+49.60	34.4969	667134.629	2192761.2	695.96	S 22°21'02.27" E
	176+63.53	34.4083	667139.938	2192773.945	695.91	S 22°08'29.78" E
	a 176+89.34	34.2705	667145.2469	2192786.691	695.87	S 21°58'31.41" E
	a 177+03.15	34.1167	667150.5559	2192799.436	695.83	S 21°58'31.41" E
	a 177+16.95	33.9628	667155.8649	2192812.182	695.80	S 21°58'31.41" E

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Title: InRoads Output - Misc. Screed Lines

Note: Elevations for screed lines are from proposed surface ML-I90WB.dtm. Lines with "Ramp" designation are from proposed surface RMP-RampA3.dtm.

Clearance Report

Report Created: 4/18/2011
Time: 6:54pm

Project: Innerbelt I-90
Description:
File Name: pw:\\HNTBW352.HNTB.ORG:DESIGN_BUILD\Documents\Walsh\49633 Central Viaduct Innerbelt WB Bridge\Design Post Award\Engineering\Modeling\Innerbelt I-90.alg
Last Revised: asteffl 4/18/2011 6:52:14 PM
Baseline Alignment: ML-I-90 WB
Input Grid Factor: 1

Note: All units in this report are in feet unless specified otherwise.

Clearance Alignment: Left EOS

Offset Point	Baseline Station	Offset	Offset Point Northing	Offset Point Easting	Offset Point Elevation	Radial Direction
	175+50.59	-45.411	667169.2375	2192638.655	699.17	S 23°50'09.93" E
	175+64.16	-45.6386	667174.9747	2192651.138	699.07	S 23°37'56.76" E
	175+77.74	-45.8638	667180.6654	2192663.642	698.96	S 23°25'43.63" E
	175+91.31	-46.0868	667186.3095	2192676.168	698.75	S 23°13'30.53" E
	176+04.88	-46.3074	667191.907	2192688.714	698.51	S 23°01'17.48" E

Clearance Alignment: Left EOS Ramp

Offset Point	Baseline Station	Offset	Offset Point Northing	Offset Point Easting	Offset Point Elevation	Radial Direction
	176+18.45	-46.5257	667197.4577	2192701.281	698.27	S 22°49'04.47" E
	176+32.01	-46.7722	667202.9859	2192713.848	698.02	S 22°36'52.03" E
	176+45.58	-47.0822	667208.5312	2192726.418	697.78	S 22°24'39.20" E
	176+59.15	-47.4467	667214.0818	2192738.985	697.54	S 22°12'26.52" E
	176+72.71	-47.8736	667219.6448	2192751.547	697.3	S 22°00'14.00" E
	a 176+98.32	-48.3485	667225.2236	2192764.101	697.07	S 21°58'31.41" E

Clearance Alignment: Crown

Offset Point	Baseline Station	Offset	Offset Point Northing	Offset Point Easting	Offset Point Elevation	Radial Direction
	175+51.92	-38.4321	667163.3964	2192642.703	699.38	S 23°48'58.11" E
	175+65.55	-38.6602	667169.1416	2192655.216	699.27	S 23°36'42.02" E
	175+79.17	-38.8859	667174.84	2192667.749	699.16	S 23°24'25.98" E
	175+92.80	-39.1095	667180.4916	2192680.304	698.95	S 23°12'09.97" E
	176+06.43	-39.3312	667186.0967	2192692.88	698.73	S 22°59'54.00" E

Clearance Alignment: Crown Ramp

Offset Point	Baseline Station	Offset	Offset Point Northing	Offset Point Easting	Offset Point Elevation	Radial Direction
	176+20.04	-39.4924	667191.5979	2192705.491	698.51	S 22°47'38.53" E
	176+33.67	-39.4914	667196.9084	2192718.194	698.28	S 22°35'22.52" E
	176+47.30	-39.5396	667202.2189	2192730.896	698.05	S 22°23'06.52" E
	176+60.92	-39.6369	667207.5294	2192743.599	697.83	S 22°10'50.54" E
	176+74.55	-39.7833	667212.8399	2192756.302	697.6	S 21°58'34.62" E
	a 177+00.22	-39.9545	667218.1504	2192769.005	697.38	S 21°58'31.41" E

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Clearance Alignment: Grade Break

Offset Point	Baseline Station	Offset	Offset Point Northing	Offset Point Easting	Offset Point Elevation	Radial Direction
	175+55.45	-19.9893	667147.9588	2192653.405	698.61	S 23°45'47.02" E
	175+69.12	-20.2176	667153.6807	2192665.896	698.5	S 23°33'28.98" E
	175+82.78	-20.4436	667159.3558	2192678.407	698.38	S 23°21'10.98" E
	175+96.45	-20.6672	667164.9839	2192690.94	698.22	S 23°08'53.02" E
	176+10.11	-20.8885	667170.565	2192703.494	698.04	S 22°56'35.11" E

Clearance Alignment: Grade Break Ramp

Offset Point	Baseline Station	Offset	Offset Point Northing	Offset Point Easting	Offset Point Elevation	Radial Direction
	176+23.77	-21.1141	667176.1051	2192716.066	697.86	S 22°44'17.24" E
	176+37.43	-21.3855	667181.6422	2192728.64	697.68	S 22°31'59.46" E
	176+51.09	-21.7135	667187.1861	2192741.21	697.51	S 22°19'41.80" E
	176+64.74	-22.1023	667192.7407	2192753.776	697.34	S 22°07'24.30" E
	a 176+90.32	-22.5546	667198.3102	2192766.335	697.16	S 21°58'31.41" E
	a 177+04.05	-23.0325	667203.8913	2192778.889	696.99	S 21°58'31.41" E

Clearance Alignment: Profile Grade

Offset Point	Baseline Station	Offset	Offset Point Northing	Offset Point Easting	Offset Point Elevation	Radial Direction
	175+59.33	0	667131.2234	2192665.006	697.78	S 23°42'17.79" E
	175+73.13	0	667136.7498	2192677.655	697.66	S 23°29'52.27" E
	175+86.93	0	667142.2305	2192690.324	697.53	S 23°17'26.75" E
	176+00.74	0	667147.6654	2192703.012	697.4	S 23°05'01.23" E
	176+14.54	0	667153.0543	2192715.72	697.26	S 22°52'35.72" E
	176+28.34	0	667158.3973	2192728.447	697.13	S 22°40'10.20" E
	176+42.15	0	667163.6943	2192741.193	696.99	S 22°27'44.68" E
	176+55.95	0	667168.9452	2192753.959	696.85	S 22°15'19.16" E
	176+69.75	0	667174.1498	2192766.743	696.72	S 22°02'53.65" E
	a 176+95.46	0	667179.318	2192779.543	696.59	S 21°58'31.41" E
	a 177+09.27	0	667184.4833	2192792.343	696.48	S 21°58'31.41" E

Clearance Alignment: Right EOS

Offset Point	Baseline Station	Offset	Offset Point Northing	Offset Point Easting	Offset Point Elevation	Radial Direction
	175+66.71	37.5	667099.8188	2192686.776	696.22	S 23°35'39.18" E
	175+80.65	37.5	667105.3225	2192699.44	696.1	S 23°23'06.01" E
	175+94.60	37.5	667110.7799	2192712.124	696.06	S 23°10'32.83" E
	176+08.54	37.5	667116.1909	2192724.828	696.02	S 22°57'59.65" E
	176+22.49	37.5	667121.5556	2192737.551	695.97	S 22°45'26.47" E
	176+36.43	37.5	667126.8737	2192750.294	695.92	S 22°32'53.29" E
	176+50.38	37.5	667132.1453	2192763.057	695.87	S 22°20'20.12" E
	176+64.32	37.5	667137.3703	2192775.838	695.83	S 22°07'46.94" E
	a 176+90.14	37.5	667142.5501	2192788.638	695.79	S 21°58'31.41" E
	a 177+03.95	37.5	667147.7173	2192801.443	695.76	S 21°58'31.41" E
	a 177+17.75	37.5	667152.8844	2192814.248	695.74	S 21°58'31.41" E

Calculations For	Innerbelt - Bridge B1 (CUY90-1627)	Job No.	49633	Sheet No.
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Backchecked by	AKS	Date	4/19/2011	1/16/2012

Title: Dead Load Deflection/Camber (@ Erection) - From CONSPAN

Beam: 1 (Exterior)	SPAN 1								
	@ 0.1L	@ 0.2L	@ 0.3L	@ 0.4L	@ 0.5L	@ 0.6L	@ 0.7L	@ 0.8L	@ 0.9L
Deck + Haunch	-0.539	-1.053	-1.457	-1.713	-1.801	-1.713	-1.457	-1.053	-0.539
DL-Prec. (DC)	-0.241	-0.470	-0.650	-0.765	-0.804	-0.765	-0.650	-0.470	-0.241
Diaphragm	-0.005	-0.010	-0.014	-0.017	-0.018	-0.017	-0.014	-0.010	-0.005
DL-Prec. (DW)	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
DL-Comp. (DC)	-0.051	-0.099	-0.137	-0.162	-0.170	-0.162	-0.137	-0.099	-0.051
DL-Comp. (DW)	-0.188	-0.368	-0.508	-0.598	-0.629	-0.598	-0.508	-0.368	-0.188
Total (in.)	-1.024	-2.000	-2.766	-3.255	-3.422	-3.255	-2.766	-2.000	-1.024
Total (ft.)	-0.085	-0.167	-0.231	-0.271	-0.285	-0.271	-0.231	-0.167	-0.085

Beam: 2 (Interior)	SPAN 1								
	@ 0.1L	@ 0.2L	@ 0.3L	@ 0.4L	@ 0.5L	@ 0.6L	@ 0.7L	@ 0.8L	@ 0.9L
Deck + Haunch	-0.590	-1.153	-1.594	-1.875	-1.971	-1.875	-1.594	-1.153	-0.590
DL-Prec. (DC)	-0.186	-0.362	-0.501	-0.590	-0.620	-0.590	-0.501	-0.362	-0.186
Diaphragm	-0.011	-0.021	-0.029	-0.034	-0.036	-0.034	-0.029	-0.021	-0.011
DL-Prec. (DW)	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
DL-Comp. (DC)	-0.054	-0.106	-0.146	-0.172	-0.181	-0.172	-0.146	-0.106	-0.054
DL-Comp. (DW)	-0.200	-0.392	-0.542	-0.637	-0.670	-0.637	-0.542	-0.392	-0.200
Total (in.)	-1.041	-2.034	-2.812	-3.308	-3.478	-3.308	-2.812	-2.034	-1.041
Total (ft.)	-0.087	-0.170	-0.234	-0.276	-0.290	-0.276	-0.234	-0.170	-0.087

Calculations For	Innerbelt - Bridge B1 (CUY90-1627)	Job No.	49633	Sheet No.
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Title: Screed Elevations

Left Edge of Slab

	SPAN 1										
	Cl. Brg. Rear	@ 0.1L	@ 0.2L	@ 0.3L	@ 0.4L	@ 0.5L	@ 0.6L	@ 0.7L	@ 0.8L	@ 0.9L	Cl. Brg. Fwd.
Deck Elevation	699.17	699.07	698.96	698.75	698.51	698.27	698.02	697.78	697.54	697.30	697.07
Dead Load Camber	0.000	0.085	0.167	0.231	0.271	0.285	0.271	0.231	0.167	0.085	0.000
Screed Elevation	699.17	699.16	699.13	698.98	698.78	698.56	698.29	698.01	697.71	697.39	697.07
Rounded Screed Elev.	699.17	699.16	699.13	698.98	698.78	698.56	698.29	698.01	697.71	697.39	697.07

Beam 1

	SPAN 1										
	Cl. Brg. Rear	@ 0.1L	@ 0.2L	@ 0.3L	@ 0.4L	@ 0.5L	@ 0.6L	@ 0.7L	@ 0.8L	@ 0.9L	Cl. Brg. Fwd.
Deck Elevation	699.27	699.18	699.07	698.86	698.63	698.40	698.15	697.91	697.67	697.43	697.20
Dead Load Camber	0.000	0.085	0.167	0.231	0.271	0.285	0.271	0.231	0.167	0.085	0.000
Screed Elevation	699.27	699.27	699.24	699.09	698.90	698.69	698.42	698.14	697.84	697.52	697.20
Rounded Screed Elev.	699.27	699.27	699.24	699.09	698.90	698.69	698.42	698.14	697.84	697.52	697.20

Crown

	SPAN 1										
	Cl. Brg. Rear	@ 0.1L	@ 0.2L	@ 0.3L	@ 0.4L	@ 0.5L	@ 0.6L	@ 0.7L	@ 0.8L	@ 0.9L	Cl. Brg. Fwd.
Deck Elevation	699.38	699.27	699.16	698.95	698.73	698.51	698.28	698.05	697.83	697.60	697.38
Dead Load Camber	0.000	0.087	0.170	0.234	0.276	0.290	0.276	0.234	0.170	0.087	0.000
Screed Elevation	699.38	699.36	699.33	699.18	699.01	698.80	698.56	698.28	698.00	697.69	697.38
Rounded Screed Elev.	699.38	699.36	699.33	699.19	699.01	698.80	698.56	698.29	698.00	697.69	697.38

Beam 2

	SPAN 1										
	Cl. Brg. Rear	@ 0.1L	@ 0.2L	@ 0.3L	@ 0.4L	@ 0.5L	@ 0.6L	@ 0.7L	@ 0.8L	@ 0.9L	Cl. Brg. Fwd.
Deck Elevation	699.15	699.04	698.93	698.73	698.52	698.31	698.11	697.90	697.70	697.50	697.29
Dead Load Camber	0.000	0.087	0.170	0.234	0.276	0.290	0.276	0.234	0.170	0.087	0.000
Screed Elevation	699.15	699.13	699.10	698.96	698.80	698.60	698.39	698.13	697.87	697.59	697.29
Rounded Screed Elev.	699.15	699.13	699.10	698.97	698.80	698.60	698.39	698.14	697.87	697.59	697.29

Beam 3

	SPAN 1										
	Cl. Brg. Rear	@ 0.1L	@ 0.2L	@ 0.3L	@ 0.4L	@ 0.5L	@ 0.6L	@ 0.7L	@ 0.8L	@ 0.9L	Cl. Brg. Fwd.
Deck Elevation	698.79	698.68	698.55	698.38	698.19	698.00	697.82	697.64	697.45	697.27	697.09
Dead Load Camber	0.000	0.087	0.170	0.234	0.276	0.290	0.276	0.234	0.170	0.087	0.000
Screed Elevation	698.79	698.77	698.72	698.61	698.47	698.29	698.10	697.87	697.62	697.36	697.09
Rounded Screed Elev.	698.79	698.77	698.72	698.62	698.47	698.29	698.10	697.88	697.62	697.36	697.09

Grade Break Line

	SPAN 1										
	Cl. Brg. Rear	@ 0.1L	@ 0.2L	@ 0.3L	@ 0.4L	@ 0.5L	@ 0.6L	@ 0.7L	@ 0.8L	@ 0.9L	Cl. Brg. Fwd.
Deck Elevation	698.61	698.50	698.38	698.22	698.04	697.86	697.68	697.51	697.34	697.16	696.99
Dead Load Camber	0.000	0.087	0.170	0.234	0.276	0.290	0.276	0.234	0.170	0.087	0.000
Screed Elevation	698.61	698.59	698.55	698.45	698.32	698.15	697.96	697.74	697.51	697.25	696.99
Rounded Screed Elev.	698.61	698.59	698.55	698.46	698.32	698.15	697.96	697.75	697.51	697.25	696.99

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HNTB

Title: Screed Elevations

Beam 4

	SPAN 1										
	Cl. Brg. Rear	@ 0.1L	@ 0.2L	@ 0.3L	@ 0.4L	@ 0.5L	@ 0.6L	@ 0.7L	@ 0.8L	@ 0.9L	Cl. Brg. Fwd.
Deck Elevation	698.58	698.45	698.31	698.14	697.95	697.76	697.58	697.40	697.22	697.06	696.81
Dead Load Camber	0.000	0.087	0.170	0.234	0.276	0.290	0.276	0.234	0.170	0.087	0.000
Screed Elevation	698.58	698.54	698.48	698.37	698.23	698.05	697.86	697.63	697.39	697.15	696.81
Rounded Screed Elev.	698.58	698.54	698.48	698.38	698.23	698.05	697.86	697.64	697.39	697.15	696.81

Beam 5

	SPAN 1										
	Cl. Brg. Rear	@ 0.1L	@ 0.2L	@ 0.3L	@ 0.4L	@ 0.5L	@ 0.6L	@ 0.7L	@ 0.8L	@ 0.9L	Cl. Brg. Fwd.
Deck Elevation	698.21	698.08	697.94	697.78	697.62	697.45	697.29	697.13	696.98	696.84	696.70
Dead Load Camber	0.000	0.087	0.170	0.234	0.276	0.290	0.276	0.234	0.170	0.087	0.000
Screed Elevation	698.21	698.17	698.11	698.01	697.90	697.74	697.57	697.36	697.15	696.93	696.70
Rounded Screed Elev.	698.21	698.17	698.11	698.02	697.90	697.74	697.57	697.37	697.15	696.93	696.70

Beam 6

	SPAN 1										
	Cl. Brg. Rear	@ 0.1L	@ 0.2L	@ 0.3L	@ 0.4L	@ 0.5L	@ 0.6L	@ 0.7L	@ 0.8L	@ 0.9L	Cl. Brg. Fwd.
Deck Elevation	697.84	697.71	697.58	697.44	697.29	697.15	697.01	696.88	696.74	696.62	696.50
Dead Load Camber	0.000	0.087	0.170	0.234	0.276	0.290	0.276	0.234	0.170	0.087	0.000
Screed Elevation	697.84	697.80	697.75	697.67	697.57	697.44	697.29	697.11	696.91	696.71	696.50
Rounded Screed Elev.	697.84	697.80	697.75	697.68	697.57	697.44	697.29	697.12	696.91	696.71	696.50

Profile Grade

	SPAN 1										
	Cl. Brg. Rear	@ 0.1L	@ 0.2L	@ 0.3L	@ 0.4L	@ 0.5L	@ 0.6L	@ 0.7L	@ 0.8L	@ 0.9L	Cl. Brg. Fwd.
Deck Elevation	697.78	697.66	697.53	697.40	697.26	697.13	696.99	696.85	696.72	696.59	696.48
Dead Load Camber	0.000	0.087	0.170	0.234	0.276	0.290	0.276	0.234	0.170	0.087	0.000
Screed Elevation	697.78	697.75	697.70	697.63	697.54	697.42	697.27	697.08	696.89	696.68	696.48
Rounded Screed Elev.	697.78	697.75	697.70	697.64	697.54	697.42	697.27	697.09	696.89	696.68	696.48

Beam 7

	SPAN 1										
	Cl. Brg. Rear	@ 0.1L	@ 0.2L	@ 0.3L	@ 0.4L	@ 0.5L	@ 0.6L	@ 0.7L	@ 0.8L	@ 0.9L	Cl. Brg. Fwd.
Deck Elevation	697.47	697.34	697.22	697.11	696.98	696.86	696.75	696.62	696.52	696.41	696.31
Dead Load Camber	0.000	0.087	0.170	0.234	0.276	0.290	0.276	0.234	0.170	0.087	0.000
Screed Elevation	697.47	697.43	697.39	697.34	697.26	697.15	697.03	696.85	696.69	696.50	696.31
Rounded Screed Elev.	697.47	697.43	697.39	697.35	697.26	697.15	697.03	696.86	696.69	696.50	696.31

Beam 8

	SPAN 1										
	Cl. Brg. Rear	@ 0.1L	@ 0.2L	@ 0.3L	@ 0.4L	@ 0.5L	@ 0.6L	@ 0.7L	@ 0.8L	@ 0.9L	Cl. Brg. Fwd.
Deck Elevation	697.10	696.97	696.86	696.78	696.67	696.57	696.49	696.38	696.30	696.21	696.14
Dead Load Camber	0.000	0.087	0.170	0.234	0.276	0.290	0.276	0.234	0.170	0.087	0.000
Screed Elevation	697.10	697.06	697.03	697.01	696.95	696.86	696.77	696.61	696.47	696.30	696.14
Rounded Screed Elev.	697.10	697.06	697.03	697.02	696.95	696.86	696.77	696.62	696.47	696.30	696.14

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Title: Screed Elevations

Beam 9

	SPAN 1										
	Cl. Brg. Rear	@ 0.1L	@ 0.2L	@ 0.3L	@ 0.4L	@ 0.5L	@ 0.6L	@ 0.7L	@ 0.8L	@ 0.9L	Cl. Brg. Fwd.
Deck Elevation	696.73	696.60	696.53	696.44	696.37	696.29	696.21	696.14	696.08	696.02	695.97
Dead Load Camber	0.000	0.087	0.170	0.234	0.276	0.290	0.276	0.234	0.170	0.087	0.000
Screed Elevation	696.73	696.69	696.70	696.67	696.65	696.58	696.49	696.37	696.25	696.11	695.97
Rounded Screed Elev.	696.73	696.69	696.70	696.68	696.65	696.58	696.49	696.38	696.25	696.11	695.97

Beam 10

	SPAN 1										
	Cl. Brg. Rear	@ 0.1L	@ 0.2L	@ 0.3L	@ 0.4L	@ 0.5L	@ 0.6L	@ 0.7L	@ 0.8L	@ 0.9L	Cl. Brg. Fwd.
Deck Elevation	696.36	696.24	696.19	696.12	696.07	696.01	695.96	695.91	695.87	695.83	695.80
Dead Load Camber	0.000	0.085	0.167	0.231	0.271	0.285	0.271	0.231	0.167	0.085	0.000
Screed Elevation	696.36	696.33	696.36	696.35	696.34	696.30	696.23	696.14	696.04	695.92	695.80
Rounded Screed Elev.	696.36	696.33	696.36	696.35	696.34	696.30	696.23	696.14	696.04	695.92	695.80

Right Edge of Slab

	SPAN 1										
	Cl. Brg. Rear	@ 0.1L	@ 0.2L	@ 0.3L	@ 0.4L	@ 0.5L	@ 0.6L	@ 0.7L	@ 0.8L	@ 0.9L	Cl. Brg. Fwd.
Deck Elevation	696.22	696.10	696.06	696.02	695.97	695.92	695.87	695.83	695.79	695.76	695.74
Dead Load Camber	0.000	0.085	0.167	0.231	0.271	0.285	0.271	0.231	0.167	0.085	0.000
Screed Elevation	696.22	696.19	696.23	696.25	696.24	696.21	696.14	696.06	695.96	695.85	695.74
Rounded Screed Elev.	696.22	696.19	696.23	696.25	696.24	696.21	696.14	696.06	695.96	695.85	695.74

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HNTB

Title: Screed Elevations

Location		Rear Abutment	0.1L	0.2L	0.3L	0.4L	0.5L	0.6L	0.7L	0.8L	0.9L	Forward Abutment
Edge of Slab (Left)	Station	175+50.59	175+64.16	175+77.74	175+91.31	176+04.88	176+18.45	176+32.01	176+45.58	176+59.15	176+72.71	a 176+98.32
	Screed Elevation	699.17	699.16	699.13	698.98	698.78	698.56	698.29	698.01	697.71	697.39	697.07
Crown	Station	175+51.92	175+65.55	175+79.17	175+92.80	176+06.43	176+20.04	176+33.67	176+47.30	176+60.92	176+74.55	a 177+00.22
	Screed Elevation	699.38	699.36	699.33	699.19	699.01	698.80	698.56	698.29	698.00	697.69	697.38
Grade-Break Line	Station	175+55.45	175+69.12	175+82.78	175+96.45	176+10.11	176+23.77	176+37.43	176+51.09	176+64.74	a 176+90.32	a 177+04.05
	Screed Elevation	698.61	698.59	698.55	698.46	698.32	698.15	697.96	697.75	697.51	697.25	696.99
Profile Grade	Station	175+59.33	175+73.13	175+86.93	176+00.74	176+14.54	176+28.34	176+42.15	176+55.95	176+69.75	a 176+95.46	a 177+09.27
	Screed Elevation	697.78	697.75	697.70	697.64	697.54	697.42	697.27	697.09	696.89	696.68	696.48
Edge of Slab (Right)	Station	175+66.71	175+80.65	175+94.60	176+08.54	176+22.49	176+36.43	176+50.38	176+64.32	a 176+90.14	a 177+03.95	a 177+17.75
	Screed Elevation	696.22	696.19	696.23	696.25	696.24	696.21	696.14	696.06	695.96	695.85	695.74

Calculations For	Innerbelt - Bridge B6 (CUY90-1627)	Job No.	49663	Sheet No.
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HNTB

Title: Final Deck Surface Elevations

Location		Rear Abutment	0.1L	0.2L	0.3L	0.4L	0.5L	0.6L	0.7L	0.8L	0.9L	Forward Abutment
Edge of Slab (Left)	Station	175+50.59	175+64.16	175+77.74	175+91.31	176+04.88	176+18.45	176+32.01	176+45.58	176+59.15	176+72.71	a 176+98.32
	Final Deck Elevation	699.17	699.07	698.96	698.75	698.51	698.27	698.02	697.78	697.54	697.30	697.07
Beam 1	Station	175+51.25	175+64.84	175+78.43	175+92.01	176+05.60	176+19.18	176+32.76	176+46.34	176+59.91	176+73.49	a 176+99.11
	Final Deck Elevation	699.27	699.18	699.07	698.86	698.63	698.40	698.15	697.91	697.67	697.43	697.20
Beam 2	Station	175+52.94	175+66.55	175+80.17	175+93.79	176+07.41	176+21.02	176+34.63	176+48.24	176+61.85	a 176+87.36	a 177+01.09
	Final Deck Elevation	699.15	699.04	698.93	698.73	698.52	698.31	698.11	697.90	697.70	697.50	697.29
Beam 3	Station	175+54.63	175+68.28	175+81.93	175+95.58	176+09.22	176+22.87	176+36.51	176+50.15	176+63.79	a 176+89.35	a 177+03.08
	Final Deck Elevation	698.79	698.68	698.55	698.38	698.19	698.00	697.82	697.64	697.45	697.27	697.09
Beam 4	Station	175+55.60	175+69.34	175+83.07	175+96.81	176+10.55	176+24.29	176+38.03	176+51.77	176+65.51	a 176+91.18	a 177+04.99
	Final Deck Elevation	698.58	698.45	698.31	698.14	697.95	697.76	697.58	697.40	697.22	697.06	696.81
Beam 5	Station	175+57.31	175+71.08	175+84.85	175+98.62	176+12.39	176+26.17	176+39.94	176+53.71	176+67.48	a 176+93.17	a 177+06.98
	Final Deck Elevation	698.21	698.08	697.94	697.78	697.62	697.45	697.29	697.13	696.98	696.84	696.70
Beam 6	Station	175+59.04	175+72.84	175+86.64	176+00.44	176+14.24	176+28.05	176+41.85	176+55.66	176+69.46	a 176+95.17	a 177+08.98
	Final Deck Elevation	697.84	697.71	697.58	697.44	697.29	697.15	697.01	696.88	696.74	696.62	696.50
Proposed Profile Grade	Station	175+59.33	175+73.13	175+86.93	176+00.74	176+14.54	176+28.34	176+42.15	176+55.95	176+69.75	a 176+95.46	a 177+09.27
	Final Deck Elevation	697.78	697.66	697.53	697.40	697.26	697.13	696.99	696.85	696.72	696.59	696.48
Beam 7	Station	175+60.77	175+74.60	175+88.43	176+02.27	176+16.10	176+29.94	176+43.78	176+57.61	176+71.45	a 176+97.16	a 177+10.97
	Final Deck Elevation	697.47	697.34	697.22	697.11	696.98	696.86	696.75	696.62	696.52	696.41	696.31
Beam 8	Station	175+62.51	175+76.37	175+90.24	176+04.10	176+17.97	176+31.84	176+45.71	176+59.58	176+73.44	a 176+99.16	a 177+12.96
	Final Deck Elevation	697.10	696.97	696.86	696.78	696.67	696.57	696.49	696.38	696.30	696.21	696.14
Beam 9	Station	175+64.25	175+78.15	175+92.05	176+05.95	176+19.85	176+33.75	176+47.65	176+61.55	a 176+87.35	a 177+01.15	a 177+14.96
	Final Deck Elevation	696.73	696.60	696.53	696.44	696.37	696.29	696.21	696.14	696.08	696.02	695.97
Beam 10	Station	175+66.01	175+79.94	175+93.87	176+07.80	176+21.73	176+35.67	176+49.60	176+63.53	a 176+89.34	a 177+03.15	a 177+16.95
	Final Deck Elevation	696.36	696.24	696.19	696.12	696.07	696.01	695.96	695.91	695.87	695.83	695.80
Edge of Slab (Right)	Station	175+66.71	175+80.65	175+94.60	176+08.54	176+22.49	176+36.43	176+50.38	176+64.32	a 176+90.14	a 177+03.95	a 177+17.75
	Final Deck Elevation	696.22	696.10	696.06	696.02	695.97	695.92	695.87	695.83	695.79	695.76	695.74

Calculations For	Innerbelt - Bridge B6 (CUY90-1627)	Job No.	49663	Sheet No.	
Made by	AKS	Date	4/18/2011	1/10/2012	
Checked by	DBT	Date	4/19/2011	1/16/2012	
Backchecked by	AKS	Date	4/19/2011	1/16/2012	

HNTB

Title: Top of Haunch Elevations

Deck Thickness: 8.5 in

Location		Rear Abutment	0.1L	0.2L	0.3L	0.4L	0.5L	0.6L	0.7L	0.8L	0.9L	Forward Abutment
Beam 1	Station	175+51.25	175+64.84	175+78.43	175+92.01	176+05.60	176+19.18	176+32.76	176+46.34	176+59.91	176+73.49	a 176+99.11
	Elevation	698.56	698.56	698.53	698.38	698.19	697.98	697.71	697.43	697.13	696.81	696.49
Beam 2	Station	175+52.94	175+66.55	175+80.17	175+93.79	176+07.41	176+21.02	176+34.63	176+48.24	176+61.85	a 176+87.36	a 177+01.09
	Elevation	698.44	698.42	698.39	698.26	698.09	697.89	697.68	697.43	697.16	696.88	696.58
Beam 3	Station	175+54.63	175+68.28	175+81.93	175+95.58	176+09.22	176+22.87	176+36.51	176+50.15	176+63.79	a 176+89.35	a 177+03.08
	Elevation	698.08	698.06	698.01	697.91	697.76	697.58	697.39	697.17	696.91	696.65	696.38
Beam 4	Station	175+55.60	175+69.34	175+83.07	175+96.81	176+10.55	176+24.29	176+38.03	176+51.77	176+65.51	a 176+91.18	a 177+04.99
	Elevation	697.87	697.83	697.77	697.67	697.52	697.34	697.15	696.93	696.68	696.44	696.10
Beam 5	Station	175+57.31	175+71.08	175+84.85	175+98.62	176+12.39	176+26.17	176+39.94	176+53.71	176+67.48	a 176+93.17	a 177+06.98
	Elevation	697.50	697.46	697.40	697.31	697.19	697.03	696.86	696.66	696.44	696.22	695.99
Beam 6	Station	175+59.04	175+72.84	175+86.64	176+00.44	176+14.24	176+28.05	176+41.85	176+55.66	176+69.46	a 176+95.17	a 177+08.98
	Elevation	697.13	697.09	697.04	696.97	696.86	696.73	696.58	696.41	696.20	696.00	695.79
Beam 7	Station	175+60.77	175+74.60	175+88.43	176+02.27	176+16.10	176+29.94	176+43.78	176+57.61	176+71.45	a 176+97.16	a 177+10.97
	Elevation	696.76	696.72	696.68	696.64	696.55	696.44	696.32	696.15	695.98	695.79	695.60
Beam 8	Station	175+62.51	175+76.37	175+90.24	176+04.10	176+17.97	176+31.84	176+45.71	176+59.58	176+73.44	a 176+99.16	a 177+12.96
	Elevation	696.39	696.35	696.32	696.31	696.24	696.15	696.06	695.91	695.76	695.59	695.43
Beam 9	Station	175+64.25	175+78.15	175+92.05	176+05.95	176+19.85	176+33.75	176+47.65	176+61.55	a 176+87.35	a 177+01.15	a 177+14.96
	Elevation	696.02	695.98	695.99	695.97	695.94	695.87	695.78	695.67	695.54	695.40	695.26
Beam 10	Station	175+66.01	175+79.94	175+93.87	176+07.80	176+21.73	176+35.67	176+49.60	176+63.53	a 176+89.34	a 177+03.15	a 177+16.95
	Elevation	695.65	695.62	695.65	695.64	695.63	695.59	695.52	695.43	695.33	695.21	695.09